SAN FRANCISCO PUBLIC LIBRARY
3 1223 07425 0912

CITY AND COUNTY OF SAN FRANCISCO
PUBLIC UTILITIES COMMISSION
UTILITIES ENGINEERING BUREAU

OCUMENTS DEPT.
SEP - 1 2003
SAN FRANCISCO
PUBLIC LIBRARY

Polhemus Bypass Conduit

Conceptual Engineering Report

5/S
ancisco Public Library
ernment Information Center
Francisco Public Library
Larkin Street, 5th Floor
Francisco, CA 94102
ERENCE BOOK
e taken from the Library

Prepared by

MANNA

Manna Consultants, Inc.
San Francisco, California

in association with
Golder Associates Inc.

July 2000

D
REF
528.15
43159c

DOCUMENTS DEPT.
SEP - 1 2003
SAN FRANCISCO
PUBLIC LIBRARY

CITY AND COUNTY OF SAN FRANCISCO
PUBLIC UTILITIES COMMISSION
UTILITIES ENGINEERING BUREAU

Polhemus Bypass Conduit

Conceptual Engineering Report

Prepared by

MANNA

Manna Consultants, Inc.
San Francisco, California

in association with
Golder Associates Inc.

July 2000

RECEIVED
AUG 3 2000
BY:

A Figures

B General Plans of Exist. PCCP

C CSBP Replacement

D [illegible]
Road [illegible]

CUW202.2
CONCEPTUAL ENGINEERING REPORT

POLHEMUS BYPASS CONDUIT

Prepared for

SAN FRANCISCO WATER DEPARTMENT

By

MANNA CONSULTANTS, INC.
SAN FRANCISCO, CALIFORNIA
IN ASSOCIATION WITH
GOLDER ASSOCIATES
UNDER DIRECTION OF THE
UTILITIES ENGINEERING BUREAU

July 2000

ROVED:

10/16/00

nald Chan — Date
nager, Civil Section – Utilities Engineering Bureau

hael E. Quan — Date
nager, Utilities Engineering Bureau

ryl Davis — Date
nager, Water Supply and Treatment Division

PROJECT TEAM

Tasso Mavroudis
Project Manager
Utilities Engineering Bureau

Subhash Shastri
Patrick Lau
Project Engineers
Utilities Engineering Bureau

Boon Lim
Construction Manager
Utilities Engineering Bureau

Jonathan Chow
Water Department
Representative

Public Utilities Commission
City and County of San Francisco
Utilities Engineering Bureau

Polhemus Bypass Conduit
CONCEPTUAL ENGINEERING REPORT
CUW 202.2
July, 2000

Table of Contents

Page

Signature Sheet
Title Page
Table of Contents

1 EXECUTIVE SUMMARY 1-1

2 INTRODUCTION
- 2.1 Purpose 2-1
- 2.2 Authorization & Scope 2-1

3 BACKGROUND & DESIGN REQUIREMENTS
- 3.1 Crystal Springs Bypass Pipeline 3-1
- 3.2 Polhemus Landslide and Effects on CSBP 3-1
- 3.3 Replacement 84-inch Buried Pipeline 3-2
- 3.4 Tunnel Alternative 3-3
- 3.5 General Design Requirements for Replacement Conduit 3-3
- 3.6 Pipe Design Considerations 3-4
  - 3.6.1 Hydraulic Design 3-4
  - 3.6.2 Structural Design 3-4
- 3.7 Tunnel Design Considerations 3-5
  - 3.7.1 Hydraulic Design 3-5
  - 3.7.2 Structural Design 3-6

4 CONCEPTUAL DESIGN ALTERNATIVES
- 4.1 General 4-1
- 4.2 Alternative 1 – Buried Pipeline with Additional Ground Support 4-1
- 4.3 Alternative 2 – Hybrid Pipeline/Tunnel with 720-ft Tunnel Section 4-1
- 4.4 Alternative 3 – Hybrid Pipeline/Tunnel with 1350-ft Long Tunnel Section 4-2
- 4.5 Alternative 4 – Tunnel 4-2

4.6 Other Considerations 4-3

5 CONSTRUCTABILITY ANALYSIS

5.1 General Constructability Considerations 5-1
5.2 Site Conditions 5-2
  5.2.1 Geology 5-2
  5.2.2 Groundwater 5-4
5.3 Trench Construction 5-4
5.4 Tunnel Construction 5-5
  5.4.1 Tunnel Cross Sections 5-5
  5.4.2 Temporary Tunnel Support 5-5
  5.4.3 Tunnel Excavation 5-6
  5.4.4 Tunnel Dewatering 5-7
  5.4.5 Tunnel Construction Sequence for Alternative 2 5-8
  5.4.6 Tunnel Construction Sequence for Alternative 3 5-9
  5.4.7 Tunnel Construction Sequence for Alternative 4 5-10
  5.4.8 Selection of Tunneling Method 5-11
  5.4.9 Spoils Disposal 5-11

5 SCHEDULE AND COST CONSIDERATIONS

6.1 Development Schedule 6-1
  6.1.1 Engineering Design 6-1
  6.1.2 Environmental Permitting 6-2
  6.1.3 Construction Schedule for Alternative 1 6-3
  6.1.4 Construction Schedule for Alternatives 2, 3 and 4 6-3
  6.1.5 Schedule Summary and Schedule Reliability 6-5
6.2 Estimated Costs for Alternatives 6-6
  6.2.1 Construction 6-6
  6.2.2 Engineering 6-8
  6.2.3 Inspection and Testing 6-8
  6.2.4 Environmental Costs 6-9
  6.2.5 Cost Reliability 6-9
    6.2.5.1 Cost Reliability of Alternative 1 6-9
    6.2.5.2 Cost Reliability of Alternative 2, 3 and 4 6-10
    6.2.5.3 TBM Construction Costs 6-10
6.3 Other Considerations 6-11

7 EVALUATION OF ALTERNATIVES

7.1 Methodology 7-1
7.2 Survivability 7-2
  7.2.1 Definitions 7-2
  7.2.2 Importance Weight 7-3
  7.2.3 Ratings of Survivability 7-3
7.3 Capital Cost 7-5
7.4 Constructability 7-5

7.5 System Redundancy 7-6
7.6 Maintenance Requirements 7-7
7.7 Time to Availability 7-8
7.8 Biological Impacts 7-9
7.9 Traffic Disruption during Construction 7-10
7.10 Public Safety during Construction 7-11
7.11 Findings 7-12
7.12 Decision Analysis Methodology 7-13

8 FINDINGS AND RECOMMENDATIONS

8.1 Summary of Findings 8-1
8.2 Recommendations 8-2

APPENDIXES

A. Figures
B. General Plans of Existing 96-Inch PCCP
C. CSBP Replacement Concepts Previously Studied
D. History of Polhemus Road Landslide and Slope Repairs
E. Field Reconnaissance for Diversion of Polhemus Road Water Transmission Pipeline
F. Polhemus Creek Alignment Option Study
G. Geology and Geologic Hazard Reconnaissance of the Slope East of Polhemus Road
H. Photographs along the Final Alignment
I. Water Inflow Into Tunnel
J. Probabilistic Seismic Slope Deformation Analyses , 84-inch Pipeline
K. Environmental Considerations
L. Discussion of SFPUC Facilities Reliability Program (FRP) Phase II, January 2000
M. Memorandum re. Valving
N. Bibliography

1. EXECUTIVE SUMMARY

3.5 General Design Requirements for Replacement Conduit

At the outset of planning, the following general design objectives were established by UEB for the replacement conduit:

- Meet the design criteria and guidelines recommended in the SFPUC's Facilities Reliability Program.
- Meet the water transmission system's hydraulic capacity requirements,
- Allow for flexibility in the system's mode of operation,

- Be structurally designed to resist internal pressures, normal external pressures, seismically induced forces, and unusual external pressures that could result from landslides;

- Maintain a high level of reliability,

- Have the least impact on the human and biological environment, and

- Have the most economic cost compatible with the above criteria.

3.6 Pipe Design Considerations

3.6.1 HYDRAULIC DESIGN

The existing 96-inch PCCP line was designed to operate under a maximum internal pressure of 230 feet of head (100 psi). This pressure condition results when the Crystal Springs Balancing Reservoir is full (Elevation 335), and the valve at the lower end of the pipeline is closed under static operating conditions (refer to transmission system network plan in Appendix L).

New pipes used in all, or a portion of, the options considered will also be designed to meet or exceed these same criteria. Actual operating condition, however, consists of a very low head, as little as under 10 feet in this particular reach of pipe.

Hydraulic design criteria have to be confirmed for outage conditions when water to Harry Tracy WTP is reversed, backfeeding to the South Bay distribution network.

3.6.2 STRUCTURAL DESIGN

The 84-inch steel replacement scheme was designed by UEB to essentially meet or exceed the seismic hazard criteria contained in the SFPUC Facilities Reliability Program.

The pipeline was designed to withstand external loads appropriate for their depth below grade, the trenching and bedding conditions specified, and the live loads to which they may be subjected as a result of their location (e.g., under a roadway). Design was also performed in accordance with current engineering practice, such as applicable American Society of Testing Materials (ASTM) and American Water Works Association (AWWA) standards.

When site-specific seismic data are used, the Seismology Committee of Structural Engineers Association of California recommend that design levels equal or exceed the level of ground motion having a 10 percent probability of exceedance in 50 years at the site [18, 21]. Design criteria for "essential" facilities, such as used for hospitals and schools, are assumed. From a seismic design viewpoint, such design criteria in the past has generally been considered to be adequately conservative because such ground movement has a low probability of being exceeded during the pipeline life.

The technical literature provides methodology to analyze and estimate ground displacements that are seismically induced and their effects on buried pipelines [22]. The literature also addresses on a probabilistic basis the occurrence of seismically induced slope ground movement. Appendix J provides a conceptual example which shows that a ground displacement of 1 foot over a 100-ft long transition zone has a probability of occurrence of approximately 2 to 5 percent in a period of 50 years. On a statistical basis, the average recurrence interval of such an event is from 500 to 2400 years.

In the case of the 84-inch steel pipeline, the pipe has been designed by UEB to withstand approximately 20 feet of overburden, and as much as 1 foot of lateral movement, over a 100-ft length of the pipe, that may result from landslide displacements.

It should be noted that more recently published probabilistic data in a preliminary report dated October, 1999, by an investigative team led by the U.S. Geologic Survey, forecasted a magnitude 6.7 Richter or greater event would have a 70 percent chance of occurrence in the next 20 to 30 years. This prediction is based on updated seismicity data and considers the aggregated effects of the San Francisco Bay Area's faults [30]. The last FRP Working Draft dated January, 2000, was essentially completed before this official announcement was made and has not considered this finding. The FRP also cited the site-specific nature of landslide hazards and stopped short of proposing specific geotechnical and structural design criteria related to landslides.

nel Design Considerations

YDRAULIC DESIGN

In accordance with the overall system demands, alternative concepts have to be designed to pass 280 cubic feet of water per second at a velocity of approximately 7 feet per second. This velocity results in relatively low friction losses. Since the existing 96-inch PCCP line has essentially the same hydraulic characteristics, the system operating parameters (valve settings, chlorination rates, etc.) will remain essentially unchanged for tunneling alternatives.

As depicted in Figure 4-1, the concept for two of the alternatives that combine segments of a shallow pipeline with segments of a tunnel include a "drop shaft" which transitions from the depth (below the roadway) of the existing pipe to the depth proposed for the tunnel sections. These drop shafts are considered necessary to accommodate the tunnel construction methods. However, they introduce secondary friction losses as the result of turbulence created in the shaft and which can be expected to persist for a considerable distance downstream.

From a hydraulic design standpoint a sloping pipeline or tunnel is preferable to a hybrid pipeline/tunnel with a vertical drop shaft. In addition to the additional minor losses for additional bends, the vertical shaft can cause air entrainment under certain flow conditions that may affect hydraulic capacity and stability of flow.

Alternatives that involve large grade drops into tunnel segments should be considered only if they can be shown to offer distinct economic advantages.

An added hydraulic consideration is the fact that the Polhemus Road pipeline, because of its location in the overall system, can possibly be called upon to operate under a wide range of delivery requirements. For example, there is the possibility that it could be called upon to deliver flow in the reverse direction under certain emergency conditions. Current investigations into these alternative operational modes should be coordinated with the hydraulic design of the tunnel. WS&TD has indicated that the full head at the Harry Tracy WTP should govern the design of the tunnel.

Appendix M discusses connection requirements between new tunnel and the existing pipeline facility.

3.7.2 STRUCTURAL DESIGN

Alternatives that include tunnel sections will be fitted with steel liners or steel pipe that are designed to withstand internal water pressure and external pressure due to groundwater.

The external pressures on the tunnel segments are calculated in accordance with current engineering practice, and tunnel bracing and lining systems are designed to withstand the overburden pressures. Actual wall pressures for tunnel segments will be estimated when a more detailed subsurface geologic investigation is performed, and a preferred tunneling method is selected.

4. CONCEPTUAL DESIGN ALTERNATIVES

4.1 General

Four conceptual design alternatives were considered for the final alternatives evaluation as follows:

- Alternative 1 – Buried Pipeline with additional ground support.
- Alternative 2 – Hybrid Pipeline/Tunnel with 720-ft tunnel section
- Alternative 3 – Hybrid Pipeline/Tunnel with 1350-ft long tunnel section
- Alternative 4 – Full-length Tunnel, approximately 3,700-ft long.

The following sections describe these alternatives. See Figure 4-1.

4.2 Alternative 1 – Buried Pipeline with Additional Ground Support

This is the buried pipeline concept that has been advanced to 85 percent design by UEB, but with the inclusion of additional ground support in the vicinity of the Polhemus landslide and existing retaining wall, and some provisions for mitigating possible hydraulic undermining resulting from a break in the existing 96-inch PCCP.

The pipe will be installed in a trench using standard trench shoring technology over its approximately 4,250-ft length. The existing retaining wall north of Ascension Drive will be strengthened with tieback anchors prior to construction, and a 100-foot segment of slope north of, and adjacent to, the existing retaining wall will be strengthened using tieback and anchors, subsurface drainage, and/or an additional new wall. Additional areas of slope stabilization on the east side of the roadway alignment may be required for this alternative, depending upon the results of a detailed subsurface investigation along the entire alignment.

4.3 Alternative 2 – Hybrid Pipeline/Tunnel with 720-ft Tunnel Section

Alternative 2 is a hybrid alternative that combines pipe-in-trench construction with tunnel construction. This alternative will follow the same alignment as Alternative 1 although the vertical grade will change in a 720-ft long tunnel section that starts at the south end of the existing retaining wall, passes the wall and a 100-ft section north of the wall, and daylights in the turnout area north of the wall. The pipe for the remaining sections of the alignment north and south of the tunnel section will be installed using trench construction. The trench will join the south end of the tunnel via a 50-ft deep vertical drop shaft. At its north end, the tunnel will join the trench using an approximately 6 percent incline.

The vertical alignment of the tunnel thus would form an inverted siphon, and the potential of ponding water in the tunnel during non-operational periods is a maintenance concern. To address this concern, a 12-inch diameter hole will be drilled from the lowest point of the tunnel northerly to the bank of the San Mateo Creek to provide drainage. A portable de-chlorination plant would be included at the outfall to treat water from infrequent drainage of the drain line.

The concrete drop-structure/transition between the pipeline portion and the tunnel portion has not been designed beyond a conceptual level. The structural design of the transition is considered to be a straightforward effort. However, the hydraulic design of the transition is considered to be challenging design issue that requires further detailed investigations.

Alternative 3 - Hybrid Pipeline/Tunnel with 1350-ft Long Tunnel Section

Alternative 3 is a hybrid alternative similar to Alternative 2 with respect to vertical and plan alignment, with the exception of the tunnel length. The tunnel segment will be 1,350-ft long, extending from the south end of the existing wall to the bank of San Mateo Creek just before the bifurcation. Like Alternative 2, a 50-ft deep vertical drop shaft at the south end of the tunnel will join the tunnel and the trench. The north end of the tunnel will daylight in the south bank of San Mateo Creek and will connect directly to the pipe-in-trench that leads to the existing bifurcation.

The structural and hydraulic design issues associated with the drop-structure/transition would be similar to Alternative 2.

Alternative 4 - Tunnel

This alternative involves constructing a 3,750-ft long tunnel to completely bypass the 96-inch CSBP and the zone of potential slope stability, located well below-ground beyond the zone of a potential deep-seated landslide. The alignment would extend from the south end of the Polhemus Lot to the bank of San Mateo Creek near its confluence with Polhemus Creek. The first 300 feet of the tunnel will be located under the small hill that forms the north flank of the Polhemus Lot, the remainder will follow Polhemus Road.

The largest remaining design problem for an all-tunnel alternative is the configuration of the tunnel cross-section (see Figures 4-2, 4-3, 4-4 for typical tunnel sections). The most economical cross-section is related to the method of tunnel construction; either by conventional mining, or by use of a tunnel boring machine (TBM). The use of a boring machine would be less costly than conventional mining, but the feasibility of its use is highly dependent upon the geologic formation encountered at tunnel depth. In order to make a selection of the best tunnel cross-section, it will be necessary to conduct a supplementary geologic exploration program.

If the TBM method of mining should prove to be feasible, the tunnel would be steel (or possibly high-density polyethylene) lined throughout its length; there would be no "free-standing", structurally-independent pipeline. A steel transition section between the tunnel lining and the existing valve structures at either end would be required.

Based on discussions between Golder and UEB during development of the tunnel concept, the tunnel cross section assumed for conventional mining consists of a 14-foot concrete-lined tunnel with a "free standing" 84-inch steel pipe passing throughout its entire length[11]. The pipeline would be supported on concrete saddle footings. There would be a three-foot annular space around the pipeline that would serve to isolate it from earth loads, and which would also provide a means for maintenance and inspection. Because this tunnel would not be a water conveying feature (as were the short tunnel sections of Alternatives 2 and 3), there would be no transition into, or out of, the pipeline, and design questions surrounding a transition structure do not arise.

It is noted that later discussions with WS&TD indicate there is no operating or maintenance need for an annular space. WS&TD prefers the tunnel, possibly steel-lined, to function as the conveyance conduit. Because of the conceptual nature of this evaluation where confirming geologic data for tunneling are still preliminary, it was determined by the consultant team that to further refine the tunnel design and costs at this time would not be appropriate.

Additional discussions of the tunnel's constructability issues are contained in Section 5.

4.6 Other Considerations

With the pipe-in-trench construction aligned in parallel with the existing 96-inch PCCP, there remains slope stabilization and ground stabilization concerns that must be addressed further in detail design. Although the additional tie-back slope strengthening has been assumed for what is believed to be the worse locations near the existing retaining structure, there may be other locations identified by more comprehensive geologic and subsurface investigations where slope strengthening may also be required.

It was reported by UEB that when the 84-inch steel pipe replacement scheme was first conceived, it was to replace the existing 96-inch PCCP. Presently however, ground stability concerns arise from the adjacency of the new 84-inch steel pipe to the existing PCCP which WS&TD now plans to keep in place as a backup conduit. There is serious concern that in the case of a break in the PCCP, there would be hydraulic undermining of the soils surrounding the 84-inch pipe, creating in effect a deteriorating "washout" condition. Mitigating measures could be developed to include advance monitoring of slide movements and pipeline joint movements, and pressure drop detection, with capability to activate upstream valve to reduce or turn off flow in the 96-inch line. Other options to consider for evaluating and mitigating such washout could include:

- Confirming that there is adequate drainage capacity, either existing or built additionally, to accommodate possible washout flow.
- Devise anchorage structures for the 84-inch steel pipeline in those lengths where it might be susceptible to washout caused by breakage of the 96-inch line.
- Examining the performance of the 84-inch steel pipeline in a "washout" condition, and its possible behavior as a "flexible conduit/cable" element.
- Constructing a concrete cutoff wall as part of an encasement structure at locations where a washout condition is most likely to occur.

5. CONSTRUCTABILITY ANALYSIS

5.1 General Constructability Considerations

The discussions in the following sections concern construction engineering issues of the alternatives. They serve as bases for the estimates of costs and schedule that are presented in Section 6.

The following constructability constraints have been considered in the conceptual design process:

- Existing retaining wall and 100-ft segment north of wall: For the pipeline alternative, in order to allow pipeline trenching in front of the wall, tie-back anchors will be installed in the wall to achieve an increased factor of safety. The section north of the wall will also require tie-backs and/or a retaining wall structure.

  For the tunnel alternative, it will be desirable to locate the tunnel outside the passive resistance zone of the wall so as not to interfere with the function of the wall. Thus, the crown of the tunnel will need to be located at minimum depths of approximately 35 and 30 feet at the south and the north end of the wall, respectively.

- Existing 96-inch pipeline and right-of-way: The pipeline must be located within the narrow corridor east of the existing 96-inch PCCP line and within the existing right-of-way.

- Traffic: It is desirable to have two-lane, two-way traffic maintained throughout construction. At the very least, one-lane traffic must be maintained with traffic control, but for short durations only. This may dictate the length of pipeline trench that can be opened at any time. Traffic constraint will also limit the choice of possible locations for the shaft, ramp, and portals in the tunnel alternatives.

- Public safety: As the pipeline trench will be excavated under busy traffic throughout construction, the shoring and bracing systems used must be designed to include traffic loads and to be stable at all times to avoid the danger and the potential disruption of traffic caused by the collapse of shoring systems. Similarly for the tunnel alternatives, the tunnel heading and excavated sections must remain stable at all times.

- Residential area: The work site is located adjacent to residential areas and night or weekend curfews may have to be observed. As such, only one work shift is planned each day for a five-day week for trench construction; and two work shifts for tunnel construction.

- Subsurface conditions: The complex geology, high ground water table, and presence of boulders are factors that strongly affect the design of trench shoring systems, selection of pipe bedding materials, and methods of excavation. The constructability of the tunnel, whether by conventional mining methods or by the use of a tunnel boring machine, is similarly affected.

5.2 Site Conditions

The following site conditions provide the bases for assessing the constructability of the alternatives, with emphasis on the tunnel's construction with its higher risk of unknown conditions. The variability of subsurface conditions and sensitivity to tunneling feasibility and costs are discussed.

5.2.1 GEOLOGY

Site geology was characterized using subsurface information developed during repair of the landslide north of Ascension Drive, and by two geologic reconnaissance investigations performed by Golder along the alignment.

The first reconnaissance was performed in March, 1998, and focussed on characterizing surface geology and geologic hazards west of Polhemus Creek in support of the Conceptual Alternatives Study. A description of this study is presented in Appendix F. The second reconnaissance was initiated in April, 2000, and focused on the slope east of the proposed alignments (east of Polhemus Road), see Appendix G. Since the geology and geologic hazards of the slope east of the alignment will have a far greater impact on the proposed alignment than will the geology of the slope west of Polhemus Creek, the following discussion focuses on the east slope. Photographs along the alignment are presented in Appendix H and referred to in the following discussion. With the exception of a 350-ft segment of the alignment at the location of the repaired landslide, no characterization of subsurface geology and groundwater has been performed along the entire 4,250-ft alignment. Subsurface information in the repaired landslide area is summarized below.

As is typical of the Franciscan Formation, the geology exposed in the road cuts along Polhemus Road varied considerably over this short, 4,250-ft section of alignment. General geologic conditions observed along the surface of the proposed alignment included: unconsolidated Quaternary Age Young Alluvial deposits composed of sand, silt, clay, gravel, and boulders; unconsolidated Quaternary Age Colluvial deposits consisting loose to firm, friable, unsorted sand, silt, clay, rock debris, and organic material; Quaternary Age Landslide deposits consisting of unconsolidated colluvial and bedrock deposits; and Jurassic and Cretaceous Age bedrock of the Franciscan Complex which was observed to consist of mostly sandstone, siltstone, greenstone, greywacke, and highly-sheared shale.

Starting with the Polhemus Lot at Station 0+00 (Photos 1 and 2) at the south end of the alignment, weathered sandstone (Photo 3) was exposed along much of the first 500 feet of the alignment. From approximately Station 3+00 to 12+75 (Photo 4), increasing exposures of lightly weathered shale (Photo 6) were observed, reverting back to weathered sandstone near the end of this segment. In general, the cut slopes in the first (southernmost) 1,500 feet of the alignment appeared stable with no apparent signs of past movements. Although traces of an ancient landslide were observed in the slope above the road in this area, it did not appear to be currently active.
The hillside east of the alignment between approximately Station 12+75 and 23+50 (Ascension Drive) appeared to contain two recent landslide areas (Photos 5, 7, and 8) separated by a boulder nest (Photo 8).

The section between Ascension Drive (Station 23+50) and the south end of the existing retaining wall (Station 28+00) also displayed characteristics of older landslides (Photo 10). Such characteristics include the presence of scarps surrounding the area and the absence of established vegetation on the slope face. The proposed shaft for Alternatives 2 and 3 will be located just south of the landslide area (Station 27+50 and the area beneath the trees in Photo 10). The slope appeared to expose boulders of weathered sandstone "floating" in a highly-weathered shale matrix.

North of the existing wall (between Stations 32+75 and 38+00), the steep cut slope east of the road was densely vegetated (Photos 11 and 12). Although there was no evidence of large-scale landsliding, numerous displays of small/shallow slope failures and soil slips were observed . Hard sandstone and chert were exposed at various locations in this reach but were commonly mantled with colluvium or debris flows. This area may be prone to landsliding due to the steepness and height of the slope. The small turnout area (left in Photo 11) corresponds approximately to the location of the inclined ramp leading to the north portal in Alternative 2.

The stream bank slopes at the San Mateo Creek exposed greenstone and weathered sandstone (Photos 13 and 14). Other than minor surficial sloughing and boulder falls, the cut slope on the east side of Polhemus Road at the north end of the alignment appeared stable. Some rock outcrops were observed near the base of the slope, although these may just be large, floating boulders. Also, many boulders, some as large as six feet in diameter, were seen scattered on the ground atop the creek bank (Photo 14) and in the creek bed.

The only subsurface data available along the alignment is from exploration and instrumentation borings drilled along the 350-ft section of the alignment adjacent to the retaining wall north of Ascension Drive. In addition, logs of the soldier pile borings drilled for the retaining wall were prepared during wall construction, Figur
5-1.

Along the face of the wall, the subsurface data available suggests that the pipeline or tunnel will be constructed in a zone of the Fransiscan Formation that consists of occasional to dense and massive lightly-weathered greenstone, sandstone, greywacke, and shale boulders "floating" in a highly sheared shale matrix. The sheared shale classifies as a lean clay with low plasticity fines and as containing fine to coarse gravel size angular shale fragments. Less weathered zones within the sheared shale have the consistency of angular gravel and cobbles.

Combining the observations from the surface reconnaissance with the limited subsurface information available at the retaining wall suggests that both the tunnel and the shallow pipeline will likely pass through highly variable geologic conditions ranging from clays to hard rocks. It is almost certain that both the tunnel and the pipeline will encounter boulders throughout the length of the alignment.

5.2.2 GROUNDWATER

In borings advanced in Polhemus Road to investigate and monitor the landslide, groundwater was observed to exist at a depth of 7 to 10 feet below the road level. Furthermore, because of the proximity of the creek, the groundwater table is likely to be close to the road level. As no groundwater studies have been performed to-date, an estimate of groundwater flow into pipe trenches and tunnel excavations could not be prepared for this study. For the tunnel, a conceptual estimate of groundwater flow was prepared by Golder and is included in Appendix I for future reference.

5.3 Trench Construction

Construction issues that will impact construction costs for pipe-in-trench construction include excavation, trench support, dewatering, and subgrade support.

As described in Section 5.2.1, trench excavation will encounter highly variable geologic conditions that will range from relatively loose fill soils to highly weathered and sheared shale, to hard graywacke and greenstone boulders and zones. It is assumed that some drilling and blasting will be required when unweathered rock is encountered during trench construction, and that excavation will be performed with hydraulic excavators.

Since some relaxation of temporary trench walls will be acceptable along most of the alignment for a proposed 15-ft deep excavation, it is assumed that the contractor installing the pipeline will use trench boxes or speed shoring to the greatest extent possible. Trench dewatering, if required, is achieved through sump pumping from the open trench. The more elaborate use of drilled dewatering wells and well points is assumed not necessary. "Dental concrete" will be required locally along much of the alignment to fill voids created from boulder removal, and from drilling and blasting.

5.4 Tunnel Construction

5.4.1 TUNNEL CROSS SECTIONS

As illustrated in Figure 4-2, the design used for relative cost estimating and scheduling adopts a horseshoe tunnel section that has finished dimensions of 13 feet (wide) by 11 feet (high) to provide a three-foot clearance on both sides and above the 84-inch pipe. The 3-ft clearance was originally specified by the UEB for pipe installation and maintenance access. Assuming a 1-ft thick concrete liner, the dimensions of the excavated section are 15 feet (wide) by 13 feet (high). The width of the excavated section is just slightly larger than that of an individual traffic lane.

The horseshoe cross section used in the current design represents the largest section that is likely to be needed, and therefore provides a conservative cost estimate. This cross section allows the tunnel to be excavated using conventional equipment. Other cross sections are possible: Figure 4-3 shows a small horseshoe section with the annular space between the tunnel walls and the walls of the pipeline grouted; and Figure 4-4 shows a circular steel-lined tunnel that carries the water directly. Detailed construction costs for these two alternative tunnel sections have not been considered in the present study because the feasibility of the sections (particularly the circular section) is dictated by the excavation method, and the excavation method is determined by the ground conditions that are largely unknown at present. Discussions with WS&TD indicated its preference for a tunnel that is steel lined, with no annular space for maintenance.

In Figure 4-4, the circular steel-lined tunnel may be excavated with a tunnel boring machine (TBM), depending on actual subsurface geologic and groundwater conditions, and whether a TBM is economically available at the time of construction. TBM's are often cost-effective where tunnels progress through uniform geologic conditions. The conditions that will likely be encountered for the proposed tunnel, alternating clay and hard rock under the groundwater table, pose the most difficult conditions for a TBM. Because of the relatively short lengths of tunnel construction in Alternatives 2 and 3, coupled with the anticipated difficult mining conditions, use of a TBM will likely not necessarily decrease the costs of Alternatives 2 and 3. A discussion about potential cost savings from a TBM for Alternative 4 is discussed in Section 6.

5.4.2 TEMPORARY TUNNEL SUPPORT

As a substantial section of the tunnel will be excavated under shallow cover (less than or just over one tunnel diameter) and live traffic, maintaining the stability of completed tunnel sections is a major safety requirement. The present study uses a temporary tunnel support system to maintain the stability of the excavated sections. The major elements of the temporary tunnel support include:

- Steel sets (such as W8x40) installed at three-foot centers;

- Steel mesh lagging between the steel sets;
- A layer of shotcrete covering the roof, sidewalls, and invert to form a closed ring; and
- "Dental treatment" to fill cavities left after removing boulders.

It is expected that the erection of the temporary tunnel support will follow closely behind the heading excavation. In particular, shotcrete will be placed at the end of each workday to support the newly excavated section and to protect the floor of the tunnel from rutting. Also, dental treatment will proceed immediately after boulder removal. For a smaller cavity (less than 18 inches deep), the dental treatment may simply be conducted by filling the cavity with one or several layers of shotcrete. For larger cavities, it may be necessary to build a bulkhead at the tunnel wall or roof to hold the backfill before filling the cavities. In this case, the bulkhead may be erected using steel mesh and lagging, and the cavities filled with pea gravel or grout.

In the absence of site-specific geotechnical information, neither the steel sets nor the shotcrete have been dimensioned. Thus, a detailed cost estimate of the temporary ground support was not prepared. Instead, a unit cost of $1,500/set or $500/foot of tunnel length (excluding labor) was assumed in the preliminary cost estimate.

4.3 TUNNEL EXCAVATION

Based on the limited rock exposure along the section of Polhemus Road over the proposed pipeline alignment and limited geologic reconnaissance, it is assumed that the tunnel will encounter the following ground conditions:

1. Sheared shale of lean clay with fragments of shaley gravels (45 percent of the tunnel length). The major tunneling concerns are (a) rutting of the tunnel floor, particularly because the tunnel is likely to be wet, (b) stability of the excavated section against squeezing ground conditions, and (c) stability of the heading and roof.
2. Weathered sandstone of clayey sand and sandy clay with large "floating" boulders (45 percent of the tunnel length). The major tunneling concerns are (a) running ground, particularly in areas with high groundwater table; and (b) excavation through boulders.
3. Rocks or room-size boulders (10 percent of the tunnel length) throughout the length of the tunnel. The major tunneling concern is rippability of the rock and boulder, and the need for drill and blast.

The assumed ground conditions suggest that, without conducting a detailed study, the best means of excavating the tunnel through the changing and unpredictable ground is to use hydraulic excavators. Boulders with diameters larger than about 4 feet that cannot be handled by the hydraulic excavator will be fragmented in place using a chemical expansion agent to allow the fragments to be removed by the hydraulic excavator. For boulders located above the springline of the tunnel, any cavities left after the removal of the boulders will be filled immediately to avoid destabilizing the roof or the wall.

For those sections of the tunnel that encounter rock or very large boulders, it is assumed that the rock will not be rippable such that the tunnel will be excavated using drill-and-blast methods. It is also assumed that the change from hydraulic excavator to drill-and-blast does not occur more than twice in the construction, and thus the delay associated with changing equipment and excavation method does not have to be separately considered in developing the tunnel advance rate. Finally, it is assumed that there is no constraint other than temporary traffic control over the period of time when blasting is permitted. If blasting were permitted to occur during a certain time window each day, then this constraint will have a significant impact on the schedule.

For those tunnel sections excavated in permeable formations (such as weathered sandstone of clayey sand) under high groundwater table conditions, a grout curtain will be installed to minimize the quantity of water inflow and thus preventing running ground conditions in the tunnel. It is assumed that the grout curtain will be installed on both the up- and down-stream side of the tunnel by drilling grout holes at 3-ft spacing to a depth of 10 feet below the invert of the tunnel. In Alternative 2, it is assumed that the grout curtain will extend the entire length of the tunnel. In Alternative 3, it is assumed that the grout curtain will be installed for a 1,000-ft section (about 75 percent of the total tunnel length). In Alternative 4, it is assumed that the grout curtain will be installed for a 2,000-ft section (just over half of the total tunnel length). The assumed length of the grout curtains are conservative and should provide a sufficient margin to cover the expected and the unexpected ground conditions.

It is possible that mechanized tunnel excavation methods that employ a roadheader, a tunnel boring machine (TBM), or in-shield boom cutters/pipe jacking may be used to expedite the excavation and thereby save costs. In particular, a TBM and pipe jacking/in-shield boom cutter can lead to cost savings for excavating small, circular tunnels.. However, as the ground conditions along the tunnel alignment are as yet largely unknown, the feasibility of using mechanized excavation methods has not been evaluated in the present study.

Because mechanized excavation holds considerable promise in being able to achieve substantial savings in construction cost and schedule, it is recommended that the feasibility of using mechanized excavation be considered in a future study if Alternative 4 is to be further evaluated.

5.4.4 TUNNEL DEWATERING

A preliminary estimate of water inflow into the tunnel during construction has been made and presented in a Technical Memorandum in Appendix I. The estimate considered only groundwater infiltration into the tunnel during and after construction, and suggested a rate of 432 gallons per day (gpd) under normal conditions and 4,320 gpd in a 3-day flush.

According to the UEB's environmental consultant, Tetra Tech, Inc., environmental permits will not be required to discharge the flow at such rates into Polhemus Creek.

The estimated water inflow considered only ground water infiltration. It did not consider the possibility of flooding of the tunnel from surface runoffs during heavy rainfalls. This is a potential problem in Alternative 2 even if sumps and pumps are installed outside the portal at the bottom of the ramp to divert the runoffs. Flooding is unlikely to be a problem for Alternatives 3 and 4.

A potential long-term post-construction source of water in the tunnel is water discharged during a blow-off operation of the pipe itself. With such operations, the chlorinated water discharged from the pipe will be de-chlorinated before being discharged into the tunnel. In Alternative 2, the de-chlorinated water can drain by gravity to the Crystal Springs Creek through the drainage gallery. For Alternatives 3 and 4, the de-chlorinated water can be discharged into the side ditch within the tunnel and drain to the north portal by gravity and eventually into the creek.

5.4.5 TUNNEL CONSTRUCTION SEQUENCE FOR ALTERNATIVE 2

In Alternative 2, the tunnel will be excavated from the ramp at the north. The short length of the tunnel and the steep grade at the ramp precludes the use of railcars or a conveyor belt system for mucking (removing excavated material from the tunnel). Thus, mucking will be removed from the heading by trucks, which will have to back from the portal into the tunnel heading. To remove the exhaust from mining and hauling vehicles, the tunnel will be fully ventilated throughout excavation. Also, as water cannot drain by gravity to the portal for this alternative, sumps will be installed at the invert at regular spacing to collect and pump the water out of the tunnel during excavation.

The installation of temporary tunnel support will follow the excavation closely. Specifically, it is assumed that no section will be left unsupported overnight. With two, 8-hour shifts a day, 5 working days a week, the assumed average advance rate is 3-ft per day. The average advance rate includes excavation, mucking, drainage, installation of temporary support, delays associated with changing equipment and excavation method, and delays associated with breaking boulders and conducting dental treatment. This average advance rate is likely to be a conservative, but attainable estimate.

Once the tunnel has been excavated and lined, a 12-inch diameter drainage gallery will be drilled from inside the tunnel using horizontal directional drilling. Conducting horizontal directional drilling from within a tunnel is a novel application that has not been attempted before. In addition to being a novel application, there are a number of other technical issues that need to be resolved for this particular application of horizontal directional drilling. For example, because the exit point of the drilling could be as much as 20 to 30 feet below the entry point, the hydrostatic pressure from the drilling fluid could cause a premature breakout before the hole breaks through the exit point. The breakout would lead to the loss in circulation of drilling fluid. Such issues are expected to result in difficult, but not insurmountable drilling conditions that would raise the cost and lengthen the schedule. Without a detailed constructability study, the cost for the drainage gallery is thus conservatively estimated at $700,000, and it is assumed that the drilling can be completed in five weeks.

The excavation of the shaft adjoining the south end of the tunnel will proceed independently of the tunnel. The shaft will be excavated within a staging area that is 50 feet long and 25 feet wide. The shaft will form telescoping sections with the upper section having a larger diameter than the lower section. The upper section will be excavated from the bottom of the trench at a distance 10 to 15 feet below the road level using a hydraulic excavator. The lower section of the shaft will be excavated from the base of the upper section, also using a hydraulic excavator. Alternatively, a straight shaft may be excavated by stationing the excavator at the working bottom of the shaft, and by removing the muck in buckets using cranes. The upper portion of the shaft is expected to be in weathered sandstone with large boulders. The lower portion of the shaft is expected to be in sheared shale. Throughout its depth, the shaft will be braced using steel frames or ring girders. When completed, the shaft will be lined with concrete. The pipeline will be installed at the center of the shaft and supported by a frame that connects to the concrete liner. As the shaft will not be used for access to the tunnel during construction, the excavation of the shaft is not on the critical path and it is assumed that the shaft can be excavated and lined in 9 weeks.

5.4.6 TUNNEL CONSTRUCTION SEQUENCE FOR ALTERNATIVE 3

For Alternative 3, the tunnel will be excavated from the north portal located on the bank of the Crystal Springs Creek. Thus, before portal excavation, the area above the bank (Photo 14 in Appendix H) will have to be graded to form a working platform and an access ramp to allow trucks to enter or exit the site to Polhemus Road.

The length of the tunnel in this alternative is 1,350 feet. At such length, the cost and schedule for mucking with either trucks backing into the tunnel or with railcars/conveyor belt are probably comparable. For the present estimate, it is assumed that tunnel muck will be transported from the heading by trucks, which will have to be backed into the tunnel. Thus, the tunnel will remain fully ventilated throughout construction. As the tunnel will have a uniform grade towards the portal, water can drain freely to the portal such that sumps and pumps will not be needed.

The installation of temporary supports will also follow the excavation closely. Again, no section will be allowed to stand unsupported overnight. With two 8-hour shifts a day, 5 working days a week, the assumed average advance rate is 4 feet per day. The average advance rate includes excavation, mucking, installation of temporary ground support, delays associated with changing equipment and excavation method, and delays associated with breaking boulders and conducting dental treatment afterwards. The average advance rate is higher than that of Alternative 2 because the open area outside the portal allows the material and equipment to move into and out of the tunnel with much greater efficiency than Alternative 2, where everything has to move through the steep ramp that exits in the middle of Polhemus Road. Another reason that the advance rate in this alternative can be higher than that of Alternative 2 is that there is no longer any need to install sumps to pump water to the portal. Again, this average advance rate is likely to be a conservative, but attainable, estimate.

Like Alternative 2, this alternative will also have a shaft in the south end of the tunnel. The shaft will be constructed using methods identical to that of Alternative 2.

5.4.7 TUNNEL CONSTRUCTION SEQUENCE FOR ALTERNATIVE 4

In Alternative 4, the tunnel will be excavated from both the north and the south portal such that the excavation will proceed from two working faces. The north portal is located adjacent to the bank of Crystal Springs Creek and will be excavated using the same method as in Alternative 3. The south portal will be located in Polhemus Lot (Photo 1 in Appendix H) where there is already a flat open area for staging and for truck and equipment movement.

The length of the tunnel in this alternative will be 3,700 feet. The south section will be excavated down-grade, and as a result the advance rate will likely be slower than that of the north section. However, as the south portal is likely to be finished first, the excavation of the south section could commence before the north section to make up for the slower advance rate. Thus, it is assumed that the tunnel excavation will be equally divided into the two sections with a length of 1,850 feet each. It is also assumed, for the sake of simplicity, that each heading will advance at the same rate.

It is assumed that, because of the length of the tunnel and the resulting economy of scale, railcars or a conveyor belt will be used for mucking. The tunnel will also remain fully ventilated at all times, as certain sections may have to be excavated by drill and blast. In the north section, water can drain freely from the heading toward the portal. In the south section, however, because the heading will always be at a lower elevation than the portal, sumps and pumps will be needed to remove water from the heading.

The installation of temporary tunnel supports will also follow the excavation closely, with no unsupported section allowed to stand overnight. With two, 8-hour shifts a day, 5 working days a week, the assumed average advance rate is 6 feet per heading per day (3 feet per heading per shift). The average advance rate includes excavation, mucking, drainage, installation of temporary ground support, delays associated with changing equipment and excavation method, and delays associated with breaking boulders and conducting dental treatment afterwards. The assumed advance rate (6 feet/day) is about 50 percent higher than that of Alternative 3 (4 feet/day). It is believed that this higher advance rate can be achieved because of several factors: the tunnel section is long enough to provide the crew with the time needed to develop an efficient work cycle, the use of a more efficient mucking system, and the availability of two headings enables the crew to pool their resources together to quicken the pace of excavation in one heading when the other heading encounters unfavorable ground conditions (such as floating boulders) and has to slow down. Again, the assumed advance rate is expected to be a conservative but attainable goal.

This alternative will also require a number of intermediate shafts to provide manhole access to the tunnel during operation. The excavation of the shafts is not on the critical path and thus will have no impact on the construction schedule.

5.4.8 SELECTION OF TUNNELING METHOD

The choice of tunnel cross sections and tunneling method may lie in the use of construction contracting procedures that have been adopted in similar circumstances -- that of allowing the bidders to propose the tunneling method.

The finished diameter of the steel lining would be specified by the Engineer, but the excavated cross section would be, within limits, left to the contractor's discretion. Such contracting procedures would allow the contractor to submit a competitive bid based upon the contractor's evaluation of the geologic conditions and upon the type of tunneling equipment available.

5.4.9 SPOILS DISPOSAL

Another design-associated problem is that of disposal of material excavated from the tunnel. Disposal of the material near the tunnel portals will probably not be a practicable option in the ecologically sensitive, residential area surrounding Polhemus Road. The location of disposal areas, and haul-away traffic programs, will have to be resolved early in the tunnel's design.

6. SCHEDULE AND COST CONSIDERATIONS

6.1 Development Schedule

Figures 6-1 through 6-4 present the development schedules for Alternatives 1 through 4, respectively. The development schedule can be broadly subdivided, for purposes of discussion, into engineering design, environmental permitting, and construction. These three stages of development are discussed below.

6.1.1 ENGINEERING DESIGN

Design of the Alternative 1 pipeline has already been completed by the UEB to about the 85 percent level and is nearly ready for issuance to bidders for construction. However, remaining design issues that must be addressed prior to construction include:

- Allocating time for a consultant to prepare a scope and budget, and for the UEB to prepare a contract, for the geotechnical investigation and final geotechnical engineering report (estimated 1 month duration). It is assumed that this work could be performed under an existing "as-needed services" agreement with the UEB. (If not, the preparation of a separate formal "Request for Proposal," preparation of competitive proposals, and a formal review and selection process, will require an additional 4 to 6 months).

- Conducting any additional detailed geotechnical investigation and laboratory testing programs for inclusion in the bid package (estimated 3 months duration).

- Completing the tie-back design for the existing retaining wall at the repaired landslide site. A tie-back design has been completed to approximately the 90 percent level and needs only to be readied as construction-ready documents (estimated 1 month duration).

- Performing slope stabilization design for the slope east of Polhemus Road along certain segments of the alignment. Localized stabilization may be required if identified during the geotechnical investigation (estimated 3 months duration, concurrent with the wall tie-back design).

- Completing preparation of construction plans, specifications, and bid documents for the pipeline to include the wall tie-back design and slope stabilization design (estimated 1 month duration).

Combining the scheduled 3 months for the site investigations, plus the 4 months for final engineering, results in a 7-month schedule for Alternative 1 Engineering Design.

Alternatives 2, 3, and 4 will all require detailed geotechnical investigations and tunnel design. Although Alternatives 2 and 3 will be able to take advantage of partial use of the nearly completed steel pipeline design, this will only marginally reduce the design schedule over Alternative 4. Accordingly, Alternatives 2, 3, and 4 were assigned similar engineering design schedules (Figures 6-2, 6-3, and 6-4 respectively). Principal schedule features assumed are:

- Because of the size of the engineering contract, outside engineering will require a formal bidding and consultant selection process (estimated 4 months duration).
- A detailed geotechnical investigation and laboratory testing programs for use in detailed design and inclusion in the bid package (estimated 3 months duration).
- Detailed design of the tunnel, including design of tunnel support and liner systems, ventilation and access shafts, and temporary and permanent dewatering (8 months duration).
- Design of tunnel to pipeline connections and revising the current pipeline design, as necessary (3 months duration concurrent with tunnel design).

To the greatest extent possible, environmental permitting will run concurrent with tunnel design. This will require that the design be staged and presented in documents that will be used for permit document preparation, such as at Conceptual (35%), Preliminary (60%), Pre-Final (85%), and Final (100%) levels of completion.

6.1.2 ENVIRONMENTAL PERMITTING

The permitting process anticipated for the project is discussed in detail in Appendix K along with detailed schedules for environmental review and agency approval.

As depicted, the permitting process is expected to take between 10 months and 16 months depending on the specific requirements under CEQA. The main discriminating factor will be the CEQA process, and more specifically, the potential requirement for an Environmental Impact Report (EIR). As described in Appendix K, CEQA is implemented in three phases. All four conduit alternatives for the new CSBP would be subject to the three phases of CEQA: (1) the first phase will consist of performing a preliminary review of the project to determine whether it is subject to CEQA, (2) the second phase involves preparing an Initial Study to determine whether the project may have a significant environmental impact, and (3) the third phase is to either prepare an EIR if the project is deemed to have significant environmental effect or to prepare a Negative Declaration (ND), or Mitigated Negative Declaration if no significant effects are found. The actual decision between an EIR and a ND remains with the Office of Environmental Review of the City and County of San Francisco.

Based on a preliminary review of the project and consultation with the City's Office of Environmental Review, it is believed that an "initial study/negative declaration" (IS/ND) may be appropriate for Alternative 1 (the steel pipeline constructed in an open trench). It is possible that that the environmental effects associated with Alternatives 2, 3, and 4 (the alternatives that include tunnel sections) may possibly trigger the need for preparation of an EIR due to issues related to the deeper excavation, spoils disposal, and impact on creek drainage. The decision for an EIR would ultimately hinge on the degree of impact determined during the preliminary phases of the assessment process.

For the conceptual purposes of this study, it was simply assumed that the schedule (10 months duration) for an IS/ND would be required for Alternative 1 and that the schedule for an EIR (16 months duration) would be required for Alternatives 2, 3, and 4.

.1.3 CONSTRUCTION SCHEDULE FOR ALTERNATIVE 1

Figure 6-1 shows the construction schedule for the steel pipeline alternative (Alternative 1). The estimated 10-month schedule includes the following main components:

- Soliciting and reviewing bids, and awarding and preparing construction contracts (4 months duration).
- Procurement of pipe and other permanent materials (4 months duration).
- Wall tieback installation and installation of additional slope reinforcement (3 months duration).
- Pipe, vault, and valve installation and connection to the existing system (5 months duration).
- Relocation of existing utilities (3 months duration concurrent with materials procurement).
- System disinfection and testing, construction clean-up, and contract closure (1 month duration).

6.1.4 CONSTRUCTION SCHEDULE FOR ALTERNATIVES 2, 3 AND 4

The estimated construction schedules for Alternatives 2, 3, and 4 ranges between 21 months and 25 months and includes the following major components:

- Soliciting and reviewing bids, and awarding and preparing construction contracts (4 months duration).

- Tunnel excavation and lining (15 to 19 months duration).

- Shaft and portal excavation (2 to 3 months concurrent with tunnel excavation).

- Grouting ahead of construction (3 to 4 months duration concurrent with tunnel excavation).

- Pipe and permanent materials procurement (5 to 6 months duration concurrent with tunnel excavation).

- Pipeline installation for Alternatives 2 and 3 (4 months duration concurrent with tunnel excavation).

- Connection of the tunnel to the pipeline for Alternatives 2 and 3 (1-1/2 to 2 months subsequent to tunnel construction).

The excavation and lining of the tunnel lie on the critical path for all three of the tunnel options. The schedule for Alternative 2 is presented in Figure 6-2. In this alternative, the excavation of the tunnel at the assumed advance rate of 3 feet/day (15 feet/week, or 60 feet/month) is expected to take 12 months. The installation of the concrete liner at the assumed rate of 200 feet/week is expected to take 4 weeks. Assuming that the installation of the concrete liner can start from the upstream end of the tunnel at the tunnel/shaft junction, the installation of the pipeline can commence three weeks after the lining starts. In this alternative, the drilling of the drainage gallery also lies on the critical path as the gallery cannot be drilled until the installation of the concrete liner has been completed. Also, the installation of the pipeline cannot proceed in the section where the drilling for the drainage gallery is in progress. Thus, the total period required for the excavation and lining of the tunnel and the drilling of the drainage gallery is 14 months. It is estimated that construction of Alternative 2, including bidding and award, will require 21 months.

The schedule for Alternative 3 is presented in Figure 6-3. In this alternative, the excavation of the tunnel at the assumed advance rate of 4 feet/day (20 feet/week, or 80 feet/month) is expected to take 17 months. The installation of the concrete liner at the assumed rate of 200 feet/week is expected to take six weeks. Assuming that the installation of the concrete liner can start from the upstream (south) end of the tunnel at the tunnel/shaft junction, the installation of the pipeline can commence two weeks after the lining starts. The total period required for the excavation and lining of the tunnel is 18 months. Alternative 3, including bidding and award, will require about 25 months to construct.

The schedule for Alternative 4 is presented in Figure 6-4. In this alternative, the excavation of the tunnel proceeds from both ends of the alignment at the assumed total advance rate of 12 feet/day (60 feet/week, or 240 feet/month with two headings), requiring 15 months. The installation of the concrete liner at the assumed rate of 400 feet/week (two headings) is expected to take eight weeks. Assuming that the installation of the pipe within the tunnel starts from the south portal and follows the installation of concrete liner by two weeks, the total period required for the excavation and lining of the tunnel is 17 months. It is estimated that construction of Alternative 4, including bidding and award, will require 24 months.

HEDULE SUMMARY AND SCHEDULE RELIABILITY

Combining engineering, permitting, and construction results in schedule durations of 21 months for Alternative 1, 37 months for Alternative 2, 42 months for Alternative 3, and 40 months for Alternative 4. While the schedule for the all-pipeline alternative (Alternative 1) contains relatively little uncertainty, there is a significant amount of uncertainty involved with development of the schedules for the tunnel alternatives, primarily with tunnel construction and permitting.

The preliminary conceptual designs for tunneling were developed over a very short time period and with limited reconnaissance level subsurface geologic and groundwater information. The objective of the preliminary conceptual design was to develop conceptual cost estimates and schedules. As such, the designs were developed on the assumption of ground conditions so unfavorable that temporary tunnel support must be installed closely behind the heading. The design also assumed that the ground conditions will be sufficiently variable that different excavation methods (hydraulic excavator and drill-and-blast) will be employed at different sections of the tunnel. The design also anticipated that the removal of large boulders and the application of dental treatment after boulder removal could further slow down the progress of the excavation. Finally, the design assumed that water inflow into the tunnel during construction could be a problem that would require the installation of a grout curtain along much of the alignment. These assumptions have served to increase the cost and schedule of tunnel construction.

There is the possibility that the ground conditions encountered in tunneling could be worse than those assumed. Worse conditions may include a higher density of "hard" boulders within the sheared shale matrix that might require frequent switching between drill-and-blast and hydraulic excavation at the tunnel face. Soft ground (soft clay or loose saturated sand) over a substantial portion of the alignment might require the use of a shielded tunnel boring machine for safe excavation. Both discoveries would lengthen the construction schedule.

It is also possible that mechanized excavation (use of a tunnel boring machine) is feasible based on more comprehensive subsurface investigation. If a machine is economically available, use of a TBM could reduce tunneling time, particularly for Alternative 4. Thus, an exploration of the subsurface conditions along the tunnel alignment and a review of the recent progress in mechanized excavation and available equipment should be conducted if the tunnel options are to be pursued further.

The schedules presented in this study should be refined after a detailed geotechnical investigation has been completed, considering also input from prospective tunneling contractors. Innovative approaches by contractors may lead to an acceleration of the schedule.

The uncertainties associated with the requirements of environmental permitting and mitigation, and of securing of easements for Alternatives 2, 3, and 4 could likewise have a significant impact on the schedule of this project. Although it has been assumed that an EIR will be required for the three tunneling alternatives, the possibility exists that the project will be granted a Negative Declaration. If an EIR is required, public input and comment can be unpredictable. Strong public opposition could cause schedule delays which would expand the schedules beyond the estimates presented.

Another impact that could adversely affect the schedules of all four alternatives would be the scheduling and coordination required to connect the new system to the existing system at both ends of the conduit. This connection will require a total shutdown of the system, for which the WS&TD of the SFPUC will allow the contractor only 48 hours. WS&TD's total shutdown time required, inclusive of contractor shutdown time, is one week. The planned shutdown can only occur during specific operational windows that occur 1 to 2 times per year. Connecting the north and south ends of the new conduit to the existing system can be staged at any period during construction with tees and butterfly valves to avoid staged tie-ins that could cause project delays.

6.2 Estimated Costs for Alternatives

Figure 6-5 summarizes the estimated development cost for each of the four alternatives. The cost estimate is divided into the four major categories of *Construction, Engineering, Inspection and Testing,* and *Environmental* costs.

Construction costs for the base option Alternative 1, the shallow pipeline, were developed by the UEB. Additional costs of the tie-back construction were developed by Golder/SOHA; and of the mitigations to hydraulic undermining were by Golder/Manna. *Engineering,* and *Inspection and Testing* costs for Alternative 1 were estimated by Golder. Tunnel development costs for Alternatives 2, 3, and 4, including *Construction, Engineering,* and *Inspection and Testing,* were developed by Golder. Tetra Tech, Inc. provided input for *Environmental* costs (exclusive of mitigation costs) for all four alternatives.

6.2.1 CONSTRUCTION

Construction costs assumed for Alternative 1 include:

- The cost for pipeline installation including: supply of the pipe, vaults, valves, and other permanent materials; trench shoring design; trenching (including shoring); construction dewatering; installation of the pipe, valves, fittings, and appurtenant structures; backfilling of the trench; pipeline testing; and pipeline disinfection. This cost was estimated by the UEB to be $8 million.

- Installation of tieback anchors in the existing retaining wall for Alternative 1. In addition, costs have been included for slope stabilization of the slope east of Polhemus Road in the 100-foot section north of the existing wall.

- Relocation of existing utilities that will be impacted by construction of the new bypass pipeline.

- Traffic control during construction.

For Alternatives 2, 3, and 4, the costs estimates for the tunnel and shaft excavation shown in Figure 6-5 are derived from a set of unit prices which Golder developed from past experience and from other published data. Detailed estimates for these alternatives are presented in Figures 6-6, 6-7, and 6-8. These unit prices are believed to be reasonably accurate order-of-magnitude figures suitable for comparison among the several alternatives. However, they may not reflect the most up-to-date pricing data for economic conditions in the San Francisco Bay Area. The estimates should not be used for budgeting purposes without further design refinement, which takes into account economic conditions at the time of construction.

Because of the short time period in which the construction cost estimate was prepared, a detailed estimate of pipeline placement in tunnel sections of Alternatives 2, 3, and 4 was not prepared. It was therefore assumed that the $8,000,000 estimated by the UEB for pipeline supply and installation for Alternative 1 could be used, with reasonable accuracy, to also represent supply and installation of the pipeline within tunnel segments of the alignment. This $8,000,000 cost for pipeline construction was subsequently added to the costs associated with the tunnel construction for these three alternatives (as summarized in Figure 6-5).

The tunnel section depicted in Figure 4-2 was used for this cost estimate as the preferred section by the UEB. Tunneling costs for Alternatives 2, 3, and 4 include all equipment, labor, and materials to construct the tunnel including grouting prior to construction to reduce water inflow into the tunnel, tunnel excavation, removal and disposal of tunnel spoils, construction of staging areas, shaft and tunnel access construction, and permanent tunnel bracing.

Alternative 2 costs include a gravity drainage gallery to permanently drain the annular space between the pipeline and tunnel walls from the low point in the tunnel.

As is seen in Figures 6-6, 6-7, and 6-8, the largest single component of tunnel construction cost is that of labor. For Alternatives 2 and 3 it was assumed that the field crew excavating the tunnel will consist of a Superintendent and Deputy Superintendent, plus a 4-person mining crew at the tunnel heading. It is assumed that tunneling will be conducted during two shifts per day. A third 4-person crew will be needed for the entry shaft construction. Based on these assumptions, approximately 626 person-weeks of labor will be required to tunnel Alternative 2, and approximately 846 person-weeks will be required to tunnel Alternative 3. At an assumed labor cost of $5,000 per person-week (including overhead and benefits), the labor costs for these two alternatives would be $3.1 million and $4.2 million respectively. These costs represent 53 percent and 64 percent, respectively, of the total tunnel construction. Alternative 4, the all-tunnel alternative using conventional-mining methods, would require 1,758 person-weeks at a total labor cost of approximately $8.8 million.

6.2.2 ENGINEERING

Engineering includes performance of a detailed subsurface geologic and groundwater investigation. Design includes all preliminary design and final design, including preparation of construction documents. In addition, the following design requirements and/or cost assumptions were assumed:

- The geotechnical investigation cost is assumed at $100 per foot of borehole drilled, which includes drilling, sampling, site supervision, laboratory testing, and preparation of geotechnical report.

- Alternative 1 will require the completion of design of a tie-back system for the existing retaining wall, design of reinforced trench shoring along segments of the alignment, design of slope reinforcement in a 100 feet segment of alignment north of the existing retaining wall, and "special structures" if required to withstand the effects of hydraulic undermining caused by a break in the adjacent 96-inch PCCP line.

- Alternatives 2, 3, and 4 will require staged preliminary and final design for the tunnel, tunnel lining, vaults, shafts, and tie-ins to the existing system. In addition, Alternatives 2 and 3 will incorporate portions of the designed pipeline depending on confirming the type of tunnel liner to be used. Accordingly, the existing pipeline drawings will need to be modified for installation in the tunnel.

6.2.3 INSPECTION AND TESTING

The cost estimate for construction *Inspection and Testing* for all alternatives includes a full time Resident Engineer throughout the duration of construction. Additional cost has been included for Alternative 1 for observations and testing of tiebacks for the existing retaining wall. Costs have been included for Alternatives 2, 3, and 4 for a Resident Tunnel Engineer team during tunnel excavation at an assumed cost of

$8,000 per week. The Resident Tunnel Engineer team would consist of a full-time resident project engineer, a full-time tunnel engineer during tunnel construction (i.e. a construction specialist on each of two daily shifts), an on-site office, and third-party testing and inspection services.

6.2.4 ENVIRONMENTAL COSTS

Environmental costs for all alternatives include preparing impact assessments, permit applications, and coordinating the approval of permits. Permit application fees are also included. Alternative 1 costs assume that a Negative Declaration will be provided by the City Office of Environmental Review. Alternatives 2, 3, and 4 assume preparation of an EIR is required. Costs presented do not include labor and expenses incurred by the UEB and the OER in support of impact assessment and permit acquisition.

The estimates also assume that no cost will be incurred in discharging groundwater from the tunnel into the creek during construction, and a nominal cost has been assigned to procure an ISND permit for the creek crossing.

It is not known if any of the alternatives will require environmental mitigation and, if so, at what cost. As such, the cost of environmental mitigation has not been included in the present estimate. It was also beyond the scope of the present study to determine if additional rights-of-way or easements will be needed for Alternatives 1 through 4. Accordingly, costs associated with procurement have not been included in the cost estimate.

6.2.5 COST RELIABILITY

The largest uncertainty in the relative cost estimate is construction cost. Variability in the other cost elements, *Engineering, Inspection and Testing,* and *Environmental,* will not significantly impact the overall cost of the project. The remainder of the discussion on cost reliability focuses, therefore, on construction costs.

6.2.5.1 COST RELIABILITY OF ALTERNATIVE 1

When compared to the costs of the alternatives that include tunnels, the cost estimated for pipeline installation in Alternative 1 is judged to be relatively reliable. However, there is a large potential for additional costs to stabilize slopes located east of Polhemus Road along certain critical segments of the alignment, or to install special mitigation measures to withstand the effects of hydraulic undermining resulting from a break in the existing 96-inch PCCP. Without the benefit of a detailed geotechnical investigation, the extent and associated costs of this additional construction cannot be estimated with accuracy. Accordingly, a $2,000,000 contingency has been added to the development cost of Alternative 1 to address the engineering design and construction additional of slope and soil stabilization measures such as tie-back anchors, rock anchors, horizontal drains, retaining walls, "special structures", etc. Because a detailed geotechnical investigation of the alignment has not yet been completed, this contingency also covers additional costs of trench dewatering during construction and difficult trenching and trench shoring as a result of variable ground conditions.

6.2.5.2 COST RELIABILITY OF ALTERNATIVES 2, 3 AND 4

A commonly used indicator for tunnel cost estimate is the cost per foot of tunnel. In the present estimate, the cost per foot for Alternatives 2, 3, and 4 are $8,150, $4,800, and $3,800, respectively. .These numbers are higher than other tunnels of comparable size. The higher costs in Alternatives 2 and 3 include the cost of the drop shaft. The high costs in all three tunnel options reflect the uncertainty associated with the need to install temporary tunnel supports closely behind the excavation, the difficulty in mucking and moving equipment and material around the congested site, and the variable geologic and groundwater conditions anticipated.

The preliminary cost analysis performed for tunnel segments of Alternatives 2, 3, and 4 suggests that shortening the period of tunnel construction could lead to cost savings. A number of possibilities exist for increasing the rate of tunnel construction. Reducing the dimension of the tunnel could shorten the schedule slightly. Allowing the installation of temporary tunnel support to lag behind the heading, or using more modern support elements such as shotcrete, or bolting in-place of the steel sets could also shorten the schedule. Finally, adopting mechanized excavation has the potential to substantially shorten the schedule. The feasibility of using mechanized excavation, the possibility of using a smaller tunnel section or alternative temporary tunnel support systems, should been investigated further in tunnel design.

6.2.5.3 TBM CONTRUCTION COSTS

To provide insight on the potential savings that could result through the use of a tunnel boring machine (TBM), Golder performed a literature search of recently constructed or recently designed tunnels that have, or will be, excavated using a TBM, as summarized in Figure 6-9. The costs presented in the table have been normalized to a unit cost per length of tunnel and per foot of diameter of the tunnel. Included are all costs associated with tunnel construction: fabrication and mobilization of the TBM; supply and installation of the tunnel lining; dewatering; and muck handling and disposal. They do not include costs associated with design, permitting, and monitoring by the Owner's representatives during construction.

Normalized unit costs range dramatically between $126 per lineal foot per foot of diameter (ft/ft) to $1,875 per ft/ft. It should be noted that the tunnels with the highest unit costs have diameters of 4 feet or less. Costs for tunnels with diameters greater than 7 feet range between $126 per ft/ft and $469 per ft/ft.

Four projects on the list have ground conditions that may be judged somewhat similar to those anticipated for the Polhemus Road tunnel: the 3,480 feet long tunnel in San Luis Obispo, California; the 1,700-ft long tunnel in Wheeling, Illinois; the 4,500-ft long tunnel in Hoover, Alabama; and the 1,000-ft long tunnel in Atlanta, Georgia. Three of these tunnels have tunnel diameters similar in size to that anticipated for the Polhemus Road tunnel. Unit costs range between $211 per ft/ft and $468 per ft/ft.

Because of the relatively short lengths of the tunnels considered for Alternatives 2 and 3, the up-front costs to construct or modify a TBM, and to mobilize and demobilize a TBM, will likely make a TBM cost prohibitive for these alternatives. Assuming a 10-ft diameter tunnel for the 3,750-ft long Alternative 4 tunnel and a range of unit costs of between $211 per ft/ft and $468 per ft/ft, construction costs would range from about $8 million to about $18 million for Alternative 4. Engineering and additional construction costs associated with the tunnel may add an additional $4 million to $5 million, resulting in total development costs of between about $12 million and $23 million. Because of the complexity of the site's subsurface geology, it is believed that the middle to upper range of this conceptual estimate provides the potential minimum cost of the Alternative 4 tunnel if a TBM is feasible. This can be compared with the Alternative 4 estimate prepared for this study considering conventional mining of nearly $25 million.

In closing, the use of a TBM is highly dependent upon the nature of the geologic structure that it must penetrate, and the economic availability of a machine that addresses project-specific needs. If, after more detailed geologic exploration is undertaken, the use of mechanized excavation is found to be applicable, it could result in a significant reduction in total project cost.

Non-conventional engineering and construction-contracting procedures that include risk/profit sharing, such as design-build contracting, may also reduce costs.

3 Other Considerations

From a design standpoint, and independent of the decision as to which alternative is to be implemented, one of the most important follow-up requirements is the need to perform additional detailed geologic exploration. The information derived from this exploration program will be required for detailed design of *any* of the alternatives, and should be implemented at the earliest reasonable date. The exploration program would consist of 12 to 14 borings along the proposed alignment (all four alternatives are on essentially the same horizontal alignment), supplemented by geophysical (sonic refraction) surveys between drill holes.

If the all-tunnel alternative is selected for implementation, the PUC's legal and administrative procedures relevant to the "contractor-selected-cross-section" concept should be evaluated immediately thereafter. Such a contracting procedure has the potential for significant cost savings, but may add three months to the implementation schedule. This additional time allows potential bidders to conduct their own supplementary geological exploration prior to preparing bids.

7. EVALUATION OF ALTERNATIVES

7.1 Methodology

Various methodologies exist in the technical literature to arrive at the selection of "preferred alternative" [20]. The Manna/Golder consultant team assisted the UEB in developing a qualitative evaluation methodology that addressed principal evaluation criteria. The criteria represent issues that, based on discussions with UEB and WS&TD, are pertinent to the selection of the preferred alternative.

Engineering issues are comprised of:

- Survivability,
- Capital cost,
- Constructability,
- System redundancy,
- Maintenance requirements, and
- Time to availability.

Environmental issues are comprised of:

- Biological impacts,
- Traffic disruption during construction, and
- Public safety during construction.

Several of these criteria are arguably subsets of others, such as traffic disruption and environmental impacts being inter-related with capital cost. But they are sufficiently sensitive (e.g., the maintenance of traffic on Polhemus Road) to warrant separate identification.

Table 1.3 (Section 1) presented a matrix that shows criteria, criteria weights, ratings and total scores. Each issue or impact is referred to as a "Rating Criterion." "Importance Weights" are assigned to each Rating Criterion based on the relative impact of that criterion to the success and goals of the project. Importance Weights, using a scale of 1 to 10, with 10 indicating the largest impact, were assigned based on consultations with WS&TD.

For each Rating Criterion, a "Rating" is assigned to each of the alternatives based on that alternative's advantages or disadvantages relative to the other alternatives. Once the Importance Weight has been assigned for each Rating Criterion, and Ratings have been assigned for each alternative, "Factors" are determined as the product of the Importance Weight and the Rating. For each alternative, the Factors are summed to determine the "Total Score." The alternative with the highest Total Score represents the most favorable alternative.

In general, the importance weight is used to express the "magnitude" of a Rating Criterion while the Rating is used to express the relative favorability between the alternatives. If the rating is considered a relative number, a numeric average must be used to weigh all Rating Criteria against. For this study, "average favorability" is defined as a Rating of 5.0 to 5.5.

7.2 Survivability

7.2.1 DEFINITIONS

"Survivability" in the technical literature has a range of definitions [14]. In one sense, "survivability" may be construed as being higher for those structures that are older and have been exposed to site-specific life history than new structures, similarly constructed, which have not undergone similar exposure and in-situ loadings. In this study, however, "survivability" is synonymous with the definition of "reliability", or specifically the probability that the CSB replacement conduit would remain serviceable over a specified period of time. The specified time is assumed to be a 50-year design life. A facility can remain serviceable beyond its design life if properly maintained.

The term "to stay in service" infers that damage could occur to the pipeline or tunnel, but the damage would not result in complete failure of the water conduit that would require immediate closure and repairs. The damage could be of a nature and extent that would allow the conduit to be repaired within a reasonable time period after the damage occurred.

Survivability considered the following possible events that could lead to catastrophic failure of the pipe:

- Age deterioration (corrosion or fatigue) of the pipe;
- Lateral deformation of the pipe by a landslide, either triggered by dynamic earthquake loads or by changes in static loading conditions;
- Overloading of the pipe by a landslide, either triggered by dynamic earthquake loads or changes in static loading conditions;
- Flooding of Polhemus Creek leading to erosion of soils around the pipeline and failure of the pipe;
- Failure of the existing 96-inch PCCP line impacting the new pipeline; and

- Shearing of the pipe by direct faulting.

7.2.2 IMPORTANCE WEIGHT

Since there is no redundancy, i.e. alternate water conveyance path, in this segment of the transmission system, it was concluded from discussions with UEB and the WS&TD that Survivability of the new bypass conduit was the single most important factor in selection of an alternative. An importance weight of 10 was consequently assigned.

7.2.3 RATINGS OF SURVIVABILITY

There is little question that a deep tunnel provides a high level of survivability because a tunnel can be installed at a sufficient depth beyond the influence of slope failures, of floods in Polhemus Creek, and of a catastrophic failure of the existing 96-inch diameter pipeline. A pipeline installed in a shallow trench could be subject to these hazards. As a result, the Rating for Alternative 4 (which uses a tunnel the entire length) should be higher than ratings for the other three alternatives that use pipe-in-trench construction.

The original goals of investigating a tunnel for this study were to increase survivability and to pass the conduit by the existing retaining wall (north of Ascension Drive) at an elevation below the influence of the existing wall to minimize the potential liability involved with trenching past the wall. Alternative 2 includes a short tunnel that would bypass the wall and the critical stability area 100 feet north of the wall, addressing liability in trenching past the wall. However, it only marginally increases the survivability of the new bypass conduit because it passes by the wall even as the purpose of the wall was to stabilize the slope in this area. Since Alternative 2 does not significantly increase the survivability of the conduit as compared to Alternative 1, their Ratings should be close in magnitude.

The tunnel in Alternative 3 was lengthened with respect to Alternative 2 primarily to allow gravity drainage of the conduit within the tunnel during pipeline draindown operations. As a side benefit, it bypasses potential landslide areas to the north of the existing retaining wall. But because it does not bypass the area where current landsliding is occurring south of Ascension Drive, the rating for Alternative 3 should be slightly greater than that for Alternative 2 but significantly lower than the Rating for Alternative 4.

Finally, the survivability of Alternative 1 (pipeline entire length) must be compared with that of Alternative 4 (tunnel entire length). Of the potential events listed above, the largest risk of potential pipeline damage is estimated to result from an earthquake-induced slope failure with a critical failure surface that could displace the pipeline laterally. Methods for analyzing the complex soil response to strong ground shaking, and its effects on steel pipe deformation, are described in the literature [22]. The pipe response is fully characterized by lateral, vertical, and longitudinal displacements over a transition length of pipe. The literature also describes means to plan and design pipelines for large ground displacements [2].

Appendix J presents a simplified study that provides a means to estimate the risk of displacement of the new 84-inch steel pipeline over the assumed 50-year design life of the project. It must be emphasized that the slope deformation results of the model discussed in Appendix J were preliminary and very sensitive to input variables such as material strengths, groundwater levels, and slope geometry, information that is not known at this time. The study presented must be updated after subsurface information along the alignment becomes available. The study is an approach that combines probabilistic seismic hazard with the probabilistic distribution of feasible subsurface conditions and properties that could be encountered in the east slopes along the alignment. The result is a relationship (Figure 3 in Appendix J) that compares the potential displacement of the pipeline with the percent risk of that displacement being exceeded in the 50-year design life of the project. Taken to a more practical outcome, the relative pipeline performance (level of damage) for a given displacement (as estimated by the UEB) was superimposed on the deformation relationship. The relative rating between Alternatives 1 and 4 can be inferred based on the relative percent chance that a limiting displacement is exceeded. Using the limiting displacement of 18 inches for functionality of the steel pipeline, a risk of exceedance of 1.3 percent (in 50 years) can be interpolated from Figure 3 of Appendix J. This can be compared with an inferred risk of exceedance of less than 0.5 percent for Alternative 4, the tunnel alternative. The difference in relative risk is small when the magnitude of risk for both alternatives is considered. Using this relationship, the relative survivability ratings between Alternatives 1 and 4 should not be extreme; with a difference on the order of 2 to 3. If the study were to indicate an order or more of magnitude difference in the probability of exceeding 18 inches, a greater difference in ratings of 6 to 8 may have been more appropriate. The close relative ratings inferred by this study must be balanced with the risk of unknown geologic conditions in the alignment of the proposed tunnel that could dramatically alter the outcome of the study to show a higher chance of exceedance.

An aspect of survivability is the reparability of the conduit upon being damaged by an earthquake or a landslide. Replacement quick-connect pipe segments can be staged in advance for emergency conditions. Given the nature of the landslide in 1997/1998 and the cross-jurisdictional issues in obtaining authorization, removal of the landslide and providing access to inspect the exterior condition of the existing PCCP was difficult. Tunnel access is considered somewhat more difficult than for pipeline. However, experience during previous earthquake events demonstrate that both tunnels and pipelines are highly reparable within days or weeks when there has been emergency response planning, including enlisting mutual aid from adjacent jurisdictions and agencies [29].

All variables considered as discussed above, a rating of 7 was assigned to Alternative 4. Alternatives 3, 2, and 1 were assigned survivability ratings of 5, 5, and 4, respectively. The ratings reflect comparative evaluation between alternatives. The Rating of 4 for Alternative 1, though noted as "below average", recognizes its acceptance in meeting the specified "structural reliability" criteria.

7.3 Capital Cost

An importance weight of 5 was assigned to Capital Cost to reflect its "medium" importance, particularly when compared to the relatively higher importance of Survivability.

This rating criterion includes all costs associated with design, permit acquisition, construction, and construction management and inspection as described in Section 6 and summarized in Figure 6-5. More specifically, capital costs considered included:

- Environmental costs for preparation of environmental documents and permit procurement. Mitigation costs are pending determination by assessments and were not included.
- Design costs including geologic characterization, pipeline and/or tunnel design, design of earth retention structures for slopes above the alignment, and preparation of construction drawings and specifications; and
- Construction costs including all labor, materials, and equipment to construct the bypass pipeline/tunnel and to relocate existing utilities as necessary; in addition to design of temporary shoring systems for trench excavation, construction dewatering, construction of earth retention structures east of the alignment; and field engineering, construction quality assurance testing, and construction management during construction.

Alternatives 2 and 3 were each assigned ratings of 5, as the $17.0 million to $17.8 million costs estimated for each of these alternatives represented approximately the average of the four alternatives. Alternative 1, with an estimated cost of $12.3 million, which includes the cost of mitigation measures for slope stabilization and hydraulic undermining, was rated 8. Alternative 4, with an estimated cost of $24.8 million, was rated 2.

7.4 Constructability

In general, Constructability is typically expressed using some of the other rating criteria, such as those that consider construction cost and schedule. In this study, Constructability addresses the risk of variability associated with the development costs and schedules due to unexpected conditions, due to availability of qualified contractors, and due to contractors' methods of construction, as compared to assumptions made in this study.

For instance, there is a higher level of confidence that the trench option can be constructed for near the cost and nearly following the schedule outlined in this study. This method of construction will not require specialized construction practices or equipment, and innovative practices by the contractor will do little to change the estimated cost or to accelerate the schedule. Conversely, tunnel construction requires a specialty contractor with specific expertise and highly specialized construction equipment. Construction costs and schedule can vary substantially from those estimated in this study depending on the contractor's methods and the availability of equipment and qualified personnel. In addition, unknown or changed subsurface geologic and groundwater conditions will have a far greater impact on the constructability of a tunnel as compared with that of open trench construction. The importance weight for this Rating Criteria represents a relative importance to the success of the project keeping to the time and budget estimates presented in this study. As the capital costs and schedules estimated for the four alternatives are considered relatively conservative, changes to the costs and schedules will likely be beneficial to the project (reduced cost and construction schedule). Because of the low resulting impact to the project, an importance weight of 2 was assigned for Constructability.

The costs and schedules estimated for pipe-in-trench segments of the alternatives have a much higher confidence than do tunnel segments. In general, confidence for accuracy of costs and schedule decrease as the length of tunnel for a given alternative increases. For this reason, Alternative 1, with no tunnel segments, was assigned a Rating of 8. Alternatives 2, 3, and 4 were assigned Ratings of 5, 4, and 3, commensurate with the decreasing level of confidence in their costs and schedules.

7.5 System Redundancy

This rating criterion compares the ability of each of the alternatives to provide redundancy to the overall transmission system. It should not be confused with survivability, which principally focuses on the reliability of the facility.

In one view, since the four alternatives considered follow the same alignment it can be said that the alternatives will provide the same relative redundancy to the transmission system. Alternatives along other alignments could provide a different level of redundancy. In this sense, this rating criteria would be assigned a lower importance weight, and all alternatives would be ranked 5.

However, system redundancy from another viewpoint is synonymous with "reliability" as it is defined in the SFPUC's Facilities Reliability Program, Phase II: Regional System Overview (FRP, Reference 16): "the ability of the *system* to meet requirements following a catastrophic event". System requirements are determined from baseline reliability characterizations that take into account hazard scenarios, system demands, facilities connectivity, and repairability. The FRP has formally identified several system improvement projects that would improve "system reliability", with the CSB being a critical component. These projects generally have to do with the ability to backfeed treated water from Peninsula reservoirs to customers in the East Bay during outage of the Irvington Tunnel, and to the South Bay during outages of the Bay Division Pipelines. The "hardening" of the CSB link appears to be part of these improvements, allowing for improved "facility reliability" while increasing overall "system reliability".

In WS&TD's systems view, system redundancy is improved with the selection of the tunnel. The CSBP is the only link in the network which is does not have alternative conveyance path that is not a tunnel. The other location where a non-redundant link occurs is the Irvington Tunnel. By constructing a tunnel for the CSB, the overall reliability of the system is improved with the CSBP link being no longer the weak link in the system.

Another factor is WS&TD's plans to leave in place the existing 96-inch PCCP even with the construction of the replacement pipeline or tunnel. The PCCP line would serve as a redundant conduit, mainly to allow planned outages of the replacement conduit (pipeline or tunnel) for purpose of repairs or maintenance. The operation of the 96-inch PCCP causes concern about a possible break which could result in hydraulic undermining of soils in the vicinity of the replacement pipeline. A replacement tunnel would be constructed deeper, away from the zone affected by such hydraulic undermining. For these reasons, the tunnel was deemed having improved system redundancy and was rated higher than the pipeline alternatives.

The importance weight of 5 reflects the average importance of System Redundancy. Alternatives 1, 2, 3, and 4 were respectively rated 3, 5, 5 and 7, to reflect the above discussions.

7.6 Maintenance Requirements

Although maintenance requirements are expected to be minor, the new pipeline and/or tunnel will require some long-term maintenance depending on the alternative selected. Such requirements could include:

- Long-term inspection of the pipe in the tunnel, if the tunnel is constructed with space between the tunnel walls and the pipeline to allow inspection;
- Maintenance of air-relief valves and butterfly valves;
- Long-term monitoring of water draining from the tunnel, if the annular space between the tunnel walls and the pipeline is not grouted;
- Possible long-term treatment of water draining from the tunnel if natural or man-made elements that occur in the water must be removed prior to surface discharge; and
- Periodic inspection of the interior of the pipe or tunnel lining (if a steel liner is used in the tunnel for water conveyance).

Maintenance requirements include costs associated with long-term operation of the tunnel/pipeline. Operating cost impacts include:

- Pipeline maintenance costs;
- Pipeline pumping costs;
- Tunnel maintenance costs;
- Costs to treat groundwater permanently discharged from tunnel options; and
- Costs to pump, drain, and treat chlorinated water from the low point in the tunnel alternatives prior to discharge in San Mateo Creek.

Other maintenance impacts that are difficult to assign costs could include:

- Accessibility for inspection;
- Accessibility for repairs;
- Long-term monitoring of water draining from tunnel;
- Risk of long-term treatment of water draining from the tunnel if natural or man-made elements that occur in the water must be removed prior to surface discharge; and
- Requirements for specialized training for inspection and repairs.

While maintenance requirements will have some impact to the project, its effect on the success of the project was considered average or slightly below average. The Water Department operates both pipelines and tunnels, so the proposed alternatives pose no great concern of additional or specialized maintenance requirements or costs. This rating criterion was accordingly assigned an importance weight of 4.

The ratings for this criterion relied principally on WS&TD's experience with the maintenance of pipes and tunnels in the existing water system. The respective ratings for Alternatives 1, 2, 3, and 4, were thus 4, 4, 4, and 8, to reflect past satisfactory performance and confidence in the maintainability of tunnel construction.

7.7 Time To Availability

This rating criterion considered the project schedule, specifically the time duration required between the present and when the bypass pipeline can be brought on-line. Schedules for each of the alternatives are discussed in detail in Section 6, and are presented in Figures 6-1 through 6-4. The development schedules considered the following:

- Preparing Requests for Proposals;
- Conducting a geotechnical investigation;

- Design and preparation of construction documents;
- Environmental reviews and document preparation;
- Procuring environmental approval;
- Construction bidding and construction contract award;
- Replacement or relocation of other impacted utilities;
- Improvements of adjacent slopes where required;
- Grouting and/or dewatering;
- Procurement of permanent materials;
- Pipeline and/or tunnel construction;
- Repairing Polhemus Road; and
- Demobilization of temporary facilities and equipment and project site clean-up.

Development schedules for the alternatives ranged between about 21 months and 42 months. The existing 96-inch bypass pipeline, still in serviceable condition, has been adequately operating for over 30 years. Assuming normal maintenance and repairs, the risk of damage to the existing pipeline was considered low over the 2 to 4 years required to put a new pipeline/tunnel on line. An importance weight of 4 was assigned to this rating criterion.

Of the four alternatives, the schedule requirement of 37 months for Alternative 2 represented the average of the four alternatives, and was therefore assigned a rating of 5. At 40 months and 42 months, respectively, the schedules for Alternatives 4 and 3 were slightly longer than Alternative 2 and were subsequently assigned ratings of 4. The substantially favorable schedule of 21 months for Alternative 1 was assigned a rating of 8.

7.8 Biological Impacts

Costs of environmental document preparation and permit procurement, as well as schedule impacts from such activities, have been included in other rating criteria. Both temporary and/or permanent biological impacts may result from construction [12]. As described in Appendix K, such impacts may include:

- Long-term seepage into the tunnel throughout the life of the pipeline. If grouting does not sufficiently reduce groundwater inflow into the tunnel, the result could be a permanent lowering of the groundwater surface along the alignment, which in turn may affect natural inflows into Polhemus Creek and plant growth on non-irrigated property adjacent to the alignment.

- The long-term increased flow in Polhemus Creek and downstream creeks resulting from the evacuation of permanent drainage from the tunnel.
- The requirement for a disposal area(s) for trench or tunnel spoils. Tunneling will create more spoils than trenching, whereas trench spoils can be used, in part, as backfill.
- The potential for siltation and sediment loading in Polhemus Creek during construction. Surface construction often creates disturbance and development of sediment (from soil stockpiles), which have the potential to wash into Polhemus Creek during rain events.
- Impacts to vegetation and the creek during construction at the creek crossing location at the north end of the alignment. Damage may include removal of vegetation for construction clearing, disturbance of the stream bed, and temporary siltation of the creek downstream of the crossing.

Although biological impacts, both during construction and during operations, are important with any project, they were judged to be low for all alternatives considered for final selection. This, coupled with inclusion of cost and schedule impacts in other rating criteria, prompted selection of an importance weight of 2 for this rating criterion.

In addition, all four alternatives will have about the same low biologic impact and were therefore each assigned a Rating of 5.

.9 Traffic Disruption During Construction

To meet the needs of San Mateo County and the local residents, all four alternatives must allow traffic to pass through the construction area, even though at a level reduced from current traffic patterns. As a result, a relatively low importance weight of 2 was assigned to this rating criterion.

Tunnel construction is estimated to last between 15 months and 18 months. Options that include trench excavation will generally have shorter construction durations but will impact road usage far more than tunnel options.

Trench excavation will dictate one-lane traffic with traffic control. Delays of as much as 10 minutes or more can be expected throughout much of the open-trench construction period. At a maximum, periods of complete road closure during open trench construction will require the public to use alternative routes. Traffic impacts for open trench options will occur during pipeline construction and during stabilization of the slopes east of Polhemus Road, where required.

Tunnel options, while requiring longer construction schedules, will impact traffic only along limited lengths of Polhemus Road at the portal and/or shafts of the tunnel. Portal or shaft access may require long-term light-controlled one lane traffic, as was used in 1998 and 1999 for the landslide. However, because of the short spans of one-lane traffic control, maximum delays will be limited to a few minutes. Grouting operations for tunnel options, if conducted from the road surface above the tunnel, will also require traffic control and short delays.

Normal traffic patterns will also be affected by the increase in truck traffic on Polhemus Road and on other arteries between the project site and the nearby freeways. Truck traffic will include trucks transporting construction equipment, temporary construction materials for trench shoring, and permanent materials such as pipe, tunnel bracing and lining systems, and trench backfill materials. Trucks will also be used to transport materials excavated from the trench or tunnel to an off-site disposal area.

Alternative 4 will provide the least impacts to traffic by a substantial margin. Portals will be constructed outside of the travelled roadway. As a result, a rating of 8 was assigned to Alternative 4. In general, impacts to traffic increase as the length of open trench for a given alternative increases. Alternatives 3, 2, and 1 were assigned ratings of 5, 4, and 3, commensurate with increasing trench length and traffic impact.

7.10 Public Safety During Construction

Similar to traffic disruption, public safety will be linked to construction methods and construction duration. Because it is the intent of San Mateo County to keep Polhemus Road open for public use to the greatest extent possible during construction, the public will be exposed to the hazards of construction. Such exposure includes:

- The presence of open trenches, both during daytime construction activities and during unguarded periods at night;
- One-lane traffic control which inherently includes an associated safety risk,
- Surface construction equipment working within feet of public traffic,
- Trucks hauling equipment and materials on Polhemus Road and adjacent roads, and
- The possible necessity of blasting in hard rock exposed in the trench excavation.

In addition, trenching may reduce the stability of existing marginally stable slopes along the alignment, possibly affecting properties at the top of those slopes.

Since construction must be conducted in accordance with OSHA and other safety regulations, construction methods for all alternatives must be adopted that inherently consider public safety. This rating criterion was therefore assigned an importance weight of 1 as not being differentiating, having little impact on the selection of the preferred alternative.

Because Alternative 4 minimizes public contact with construction operations to a greater extent than the other alternatives, it was assigned a rating of 7. A rating higher than this was not assigned due to the duration of this alternative and the consideration of construction traffic for the extended period of time. As the length of trenching increases in each of the other alternatives, so does public contact and safety concerns. Alternatives 3, 2, and 1 were assigned Ratings of 5, 5, and 4, respectively, commensurate with the increasing open trench length for theses alternatives.

7.11 Findings

The weighted scores ranked the all-tunnel Alternative 4 as the preferred alternative (Table 1.3). The pipeline (Alternative 1) ranked next highest, followed by the hybrid alternatives (Alternatives 2 and 3). Factors favoring the tunnel over the pipeline are:

- Higher reliability and survivability due primarily to its location outside the zone affected by potential deep-seated landslides and potential failure of the existing 96-inch PCCP which WS&TD plans to retain as a backup during planned outages of the new bypass conduit.
- Greater ability to remain in operation despite some earthquake damage.
- Lower risk of loss of life or damage to property resulting from breaks in the 96-inch PCCP, or damage in the tunnel.
- Less risk of complications and delays due to multiple jurisdiction of roadway and adjacent property during an emergency response.
- Less impact on traffic during construction.

Factors favoring the pipeline over the tunnel are:

- Lower cost,
- Available sooner.

The hybrid alternatives did not offer sufficient benefits to offset the additional costs of the partial length tunnel and drop structure construction.

7.12 Decision Analysis Methodology

The matrix of weighted criteria and ratings developed in this section and shown in Table 1.3 (Section 1) is but one of several approaches that are available for arriving at a determination of the preferred alternative. The actual methodology employed is merely a "decision tool" with which individuals, or groups of individuals, can arrive at a basis for a decision. In a group setting, or inter-departmental setting, arriving at a consensus decision is not always possible given differing agenda and priorities and personal viewpoints. Computerized decision analysis algorithms are available to help to identify causal patterns of criteria, priorities, and outcome, or "decision trends". Two programs examined for this report are *Expert Choice 2000*, and *Avantos' Decide-Right* (1999), selected because of their simplicity of use and user-friendly procedures and screen displays. (Appendix L provides the findings derived from applying the *Avantos* software to the Polhemus Bypass Conduit project).

Common to these programs is the recognition that "qualitative" decision making is a natural process. They recognize that ratings are naturally analog concepts, and have incorporated into the procedures qualitative measures of priorities and criteria weights (e.g., "Low importance", "high importance", etc.). Ratings, similarly, could be made qualitatively rather than with numbers. Composite ratings and scores are computed using median and normalizing principles, not just numerical averages, so that there is mathematical and statistical basis to the scoring.

In one case, the computer program recognizes that groups may have differing views on criteria, criteria weights, and ratings, and therefore has provided forms for use by individuals for input into the program. The program itself would analyze the group's samplings and display the group's results, showing trends and decision preferences. Most interestingly, this program allows individuals to also weigh one criteria against any other criteria subjectively. In these instances, the program calculates an "inconsistency" rating that shows the extent of deviation of these weightings from syllogistic relationships. This approach thus recognizes that natural decision making has inherent human factors that may be difficult to express solely in terms of engineering criteria. The program itself does not make the decision. Its usefulness is in facilitating group discussion, identifying areas of agreement and disagreement, and helping to develop a consensus outcome that is defensible.

The ultimate usefulness of a decision analysis procedure is to provide a documented means to scrutinize the bases for capital spending decisions. In the instance of the Polhemus Bypass Conduit project, it is quite evident that WS&TD, in its emphasis on Survivability over Cost, desires a high degree of reliability in the replacement facility. At the core of any economic justification for choosing the more reliable but more costly tunnel alternative are the benefits that accrue from the increased reliability. The FRP Phase II document has proposed a package of risk reduction projects for "early implementation". The justification of constructing the more costly tunnel rests with this total regional 'hardening' proposal. Outages would have significant socio-economic impacts to the region. A decision to build the tunnel, and establishing the parameters for its design, are therefore linked to the coordinated and parallel completion of findings (and eventual decisions and implementation) on the other projects in the package, i.e. Harry Tracy WTP backfeed, Pulgas Tunnel pressurization, valve and appurtenance upgrades elsewhere, etc.

8. FINDINGS AND RECOMMENDATIONS

8.1 Summary of Findings

Even with the construction of the new replacement conduit, whether a tunnel or the 84-inch steel pipeline, WS&TD intends to keep and operate the existing 96-inch PCCP as a redundant conduit to enable shutdown of the new conduit for maintenance or inspections. WS&TD's view is that, given the Crystal Springs Bypass' (CSB) critical link, there is unacceptable risk of seasonally recurring threats to the integrity of the PCCP. Such renewal schemes as slip-lining the existing PCCP do not provide the additional structural capacity to resist the effects of landslides.

Installing a new steel pipeline along and in the proximity of the PCCP leaves it prone to similar landslide threats. The new steel pipeline can theoretically be designed to itself withstand the effects of large ground displacements and surcharge that's likely to be encountered in a landslide or an earthquake. However, there is abiding concern that a break in the adjacent PCCP line will cause hydraulic undermining of the bedding and soils surrounding the steel pipeline.

Mitigation measures, such as providing cutoff wall structures along the steel pipe, or providing anchorage to allow the steel pipe to span between supports in the event of undermining, could be devised and installed. However, these mitigation measures do not provide the certainty and reliability that WS&TD desires for this critical "lifeline" link in the water system. WS&TD's concern rests largely on the experience of the 1997/98 landslide where operations were continued under constant monitoring amid the threat of outage from a burst PCCP line. Additionally, access to inspect and repair any damage to the PCCP line was hampered by the difficulty of removing the landslide earth mass, and by cross-jurisdictional issues that delayed response time.

To physically clear the threat of landslides, the replacement conduit has to be aligned away from the zone of a potential slide. The replacement conduit has to either be a tunnel placed deeper below ground, or a new steel replacement pipe that follows a plan alignment located at a significant distance away from any threat of slope stability.

The UEB's current design of the replacement 84-inch steel pipeline was originally based on providing an "interim fix" amid the prevailing emergency condition at the time following the landslide occurrence. The intention, prior to inspections that indicate the 96-inch PCCP was partly damaged but still serviceable, was also to replace the PCCP line, putting it out of service. "Quick fixes" consisting of merely pipe replacement of damaged segments would have been economic but were rejected from the outset because they would not increase the reliability of the existing PCCP. "Long-term fixes" involve the construction of a new pipeline or a tunnel along another alignment such as El Camino Real, or other alignments easterly of the San Mateo Highlands located away from unstable slopes, but permitting challenges and resulting delays put implementation too far into the future. The "interim fix" approach decided by the SFPUC/UEB was considered at the time a cost-effective solution to achieve a reliability criterion that's principally focused on structural performance of the steel pipe under seismic and surcharge loads.

For this CER preparation, reliability criteria were established with WS&TD's input, and in conjunction with published system operating goals contained in the SFPUC Facility Reliability Program, Phase II: Regional System Overview (FRP) [16]. In this system context, the CSB serves a vital link in an inter-connected network, which can be operated flexibly to respond to varying system demands and emergency outages. On this qualitative basis, the tunnel was deemed more reliable than the steel pipe for several reasons: (1) it is physically located outside the zone of potential slide and influence of hydraulic undermining caused by a break in the 96-inch PCCP; (2) the tunnel, when damaged in a landslide or an earthquake, does not usually result in outage; (3) the tunnel's construction, when coupled with other currently proposed system improvements, will allow WS&TD to operate its contingency plan of reversing flow from the City's reservoirs to the Peninsula and South Bay distribution system; and (4) in WS&TD's expressed view, constructing a tunnel replacement at the CSB link increases its reliability to a level comparable with the Irvington Tunnel link which is the only other critical, non-redundant link in the regional transmission network.

In the alternatives considered, the difference in cost between the pipeline and the tunnel was estimated to be $12.5 million. This difference could be reduced if it is shown that the use of a tunnel boring machine (TBM) is feasible given the site's bouldery Franciscan "melange". The significant difference in cost between constructing a tunnel and installing a pipeline is economically equivalent to the present worth of annualized costs of assumed risk reduction.

The benefit/cost analysis to justify such a capital improvement should be performed on a "system" basis. Such an analysis was not in Manna/Golder's scope, but could be done in accordance with guidelines established in the FRP for follow-on Phase III prioritization and implementation activities to seismically harden SFPUC's regional water supply network. The main benefit of the increased reliability of a tunnel is enhanced system performance, but there are also socio-economic benefits that accrue from a more reliable installation.

Besides WS&TD, implementation of the tunnel project has to also consider the input and participation of the SFPUC's water customers represented by BAWUA.

8.2 Recommendations

These recommendations result from this conceptual design study:

1. **UEB should proceed with field investigations, planning and design of a new tunnel.** The tunnel would generally parallel the alignment of the existing 96-inch PCCP, but comprehensive geologic investigations and engineering analysis are needed to determine the best

overall alignment and cross section. An optimum tunnel cross section design, besides meeting hydraulic criteria, should consider constructability issues. The feasibility of use of a tunnel boring machine, if confirmed by geologic investigations and contractor availability, may result in cost savings over conventionally mined methods. Design of the tunnel, tunnel lining and appurtenances, should consider the tunnel's operating conditions, and pressurized flow and connectivity with an upgraded Pulgas Tunnel/Crystal Springs Tunnel contemplated in SFPUC's master plan. Advance construction of the wye that connects the new tunnel to the existing 96-inch PCCP could occur as phase one of the tunnel construction.

2. **UEB should initiate environmental studies and acquire permits for the tunnel alternative** Under the California Environmental Quality Act (CEQA), the City as lead agency is required to approve the project and prepare CEQA documents. Significant issues would include presence of rare or endangered plants or animals, potential impacts to soils or land stability, potential impacts to water quality, and fish habitat, historic and cultural resources, and potential impacts to land use and traffic flow. Environmental constraints may impact specific siting of facilities such as tunnel adits, spoil disposal areas and construction laydown areas. The completion of impact assessments and the determination of any required mitigations will help confirm the project cost and schedule.

Orderly project planning requires having in place approved written operating plans. Although it contains important operating and system outage plan details that are not yet confirmed by the SFPUC, the FRP Working Draft provides nonetheless a useful framework for master planning system-wide improvements. As part of the ensuing Phase III of the FRP, the SFPUC should therefore confirm operational, emergency response and economic planning criteria for upgrading of specific facilities to meet the goals of the FRP.

A key component of such planning, as alluded to in the FRP, is the consideration of the reliability of the SFPUC's wholesale water customers' (BAWUA) distribution subsystem. In selecting the higher cost tunnel alternative for the Crystal Springs Bypass, the SFPUC is seeking to achieve a high level of reliability commensurate with the CSBP conduit being a critical "lifeline" which connects to BAWUA's distribution systems. It is therefore reasonable to expect that discussions between the SFPUC and BAWUA would include inter-agency emergency response operational planning, as well as project cost and equity share. In any event, the design and construction of the tunnel project should have the benefit of BAWUA's input and review.

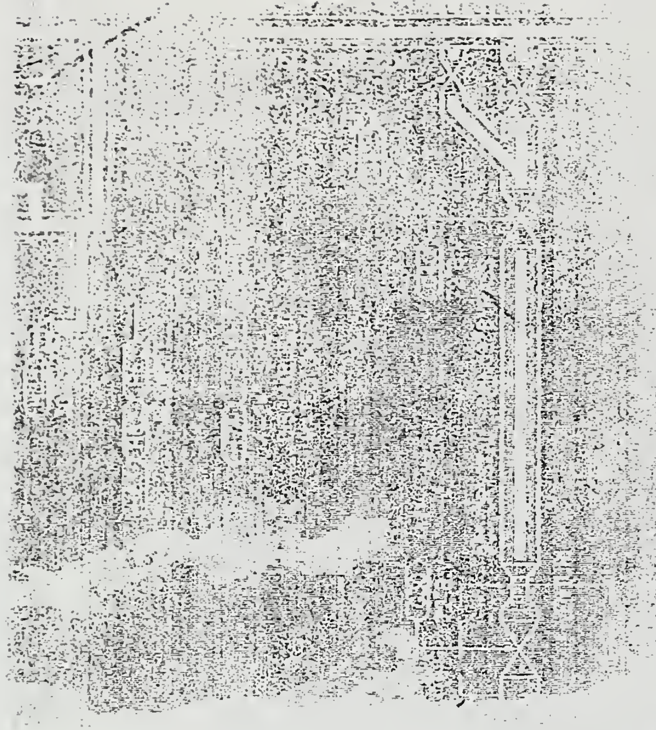
Design investigations of certain recommended FRP Early Improvement Projects, such as the pressurization of the Pulgas Tunnel, the backfeeding of Harry Tracy

WTP water, and Capuchino Valve Lot improvements, should be coordinated with the planning and construction of the new tunnel.

The WST&D has expressed its intention to maintain operations of the 96-inch line, principally as a redundant conduit when flow in the tunnel has to be shut down, such as during maintenance and inspections. The SFPUC should therefore confirm future operating and maintenance plans for the existing 96-inch PCCP line during the construction of the replacement tunnel and after tunnel construction is complete. Operational planning should follow guidelines established in the FRP, with clarification of what's deemed acceptable response and preparedness for a seismic event, or a site-specific landslide occurrence. Removal of the rock fill in Polhemus Creek constructed in March 1998 to buttress against further slide damage to the PCCP line, if required as part of creek restoration, should be staged in accordance with environmental clearance conditions and the tunnel construction schedule.

Finally, underlying the recommendations presented in this report is the premise that the SFPUC will schedule, coordinate and expedite all activities important to funding and implementing the tunnel's design and construction. The FRP's "early implementation" projects, some of which have been improvements long planned within the SFPUC departments should be accelerated to an early conclusion because their findings are critical input to the design of the tunnel. Conservative design assumptions, such as establishing the tunnel's hydraulic gradient, may be made in order to avoid delaying design activities. Design assumptions should be made to not preclude future options contemplated in the reliability improvement master plan.

Another possible schedule issue is project funding. Some funding for the PCCP pipeline replacement was available for the steel pipeline scheme. During the course of this CER preparation, the WS&TD's management stated that it expects funding of the costlier but more reliable tunnel to be achievable. Negotiations to obtain project funding will benefit greatly from the high degree of reliability of the tunnel against outage, and the bypass conduit's critical water quality function. Current investigations of system reliability, outage planning, and the BAWUA distribution network' s requirements, have to be coordinated with the tunnel's implementation.



CITY AND COUNTY OF SAN FRANCISCO
PUBLIC UTILITIES COMMISSION
UTILITIES ENGINEERING BUREAU

Polhemus Bypass Conduit
Conceptual Engineering Report

Prepared by

MANNA

Manna Consultants, Inc.
San Francisco, California

in association with
Golder Associates Inc.

July 2000

Appendix A

Figures

Appendix B

General Plans of Existing 96-inch PCCP

Appendix C

CSBP Replacement Concepts Previously Studied

Appendix C

CSBP Replacement Concepts Previously Studied[1]

Prior to designing the 84-inch CSBP Replacement Pipeline, the SFPUC/UEB had considered alternative alignments, and short-term, medium-term, and long-term options. The concepts considered are depicted schematically in Figures C-1 through C-3. Table C-1 summarizes the assessments of the various options considered by UEB.

3. 1 Short-Term Options (QF-1, QF-2)

Short-term options centered on relying on the existing PCCP line and replacing either damaged sections or sections that could become damaged through future landslides with welded steel pipe. These short-term options would be more economical than longer-term alternatives. As presented in Table C-1, two alternatives were investigated. Option QF-1 (QF, for "quick fix") focused on replacing about 60 feet of the PCCP line adjacent to the repaired landslide where distress to the pipe was observed to be greatest after the landslide occurred. For Option QF-2, a new steel pipe would replace the existing PCCP pipe between Stations 28+00 and 36+50 along a new alignment located on the west side of Polhemus Creek. These alternatives were eliminated from further evaluation because they provide a higher level of reliability for only a small portion of the alignment, leaving the remaining PCCP line still vulnerable to landslide hazard damage. In addition, repairs to only portions of the PCCP line would be inconsistent with the SFPUC's policy of replacing all large-diameter prestressed concrete pipe presently in the transmission system.

3.2 Medium-Term Options

Several options focused on constructing a new pipeline in the Polhemus Creek valley that would parallel the existing alignment. These options were considered medium-term ("intermediate fix" or IF) options in that they would provide a greater level of reliability when compared with the short-term options, but not as high a level of reliability as alternatives that followed alignments remote from the Polhemus Creek valley.

---

[1] Compiled by Manna/Golder, based on sketch, verbal, and noted information furnished by UEB [27].

More specifically, advantages of medium-term options over short-term and long-term options included:

- Higher reliability over short-term options,
- Generally faster development schedule than long-term options as a result of simpler design, rapid right-of-way procurement, and shorter construction duration; and
- Generally lower development costs than long-term options.

The main disadvantage common throughout the medium-term options is their exposure to the same geologic hazards as the existing 96-inch PCCP line. To reduce the risk of damage under these hazard conditions, it was decided that the medium-term options would use a welded steel pipe that could withstand significantly greater ground displacement than the existing PCCP pipe.

3.2.1 Pipeline Below Centerline of Polhemus Road (IF-1)

The alternative adopted for final design of the UEB involved constructing the new waterline adjacent to the existing 96-inch waterline under Polhemus Road. The centerline of the new pipeline would be located near the centerline of Polhemus Road at a sufficient distance from the existing pipeline to limit disturbance to the existing pipeline during construction. It would also be constructed with an invert elevation slightly higher than that of the existing pipeline to limit disturbance to the ground support of to the existing pipeline.

Positive aspects of this alignment include:

- Very good hydraulic characteristics (low friction losses) achieved by the minimizing of bends and fittings.
- Ease of construction, as construction will be performed on flat ground with excellent access and could be performed in any season, resulting in lower construction costs and a shorter construction schedule.
- Low (almost insignificant) biological impacts.
- Minimal schedule impacts due to low biological environmental impacts.

- This alignment is generally beyond the influence of most stability concerns that could arise from the west valley wall. However, it could be impacted from instability or low stability of the east valley wall.

The UEB considered this alternative a viable alternative worthy of further study, and ultimately became the pipeline Alternative 1 that is one of the four alternatives considered in this Conceptual Engineering Report. In selecting this alternative for design, the negative aspects were considered by UEB at the time as mitigable impacts that would not cause significant delay or add significant cost to the project. Negative aspects include:

- Greater traffic impacts than other medium-term options.
- The requirements of permanent retention systems to increase stability of the slope east of Polhemus Road along portions of the alignment.
- Unresolved geotechnical stability issues for adjoining properties resulting from the temporary excavation for the pipeline and/or construction of permanent retention systems.

In particular, concerns have been raised of the impact of pipeline construction to the wall and retention system that was constructed in 1998 and 1999 to repair the landslide north of Ascension Drive, and a 100-ft segment of the alignment north of the wall. The centerline of the new pipeline would be installed about 20 to 24 feet from the east face of the existing wall. The temporary excavation for the new waterline, which could extend about 15 feet below Polhemus Road, would reduce passive resistance mobilized by the existing retaining system. Tieback anchors to accommodate this decrease in passive resistance have been designed by SOHA Engineers and Golder [19].

Golder and Manna, starting in April 2000 in a separate task order, conducted a geotechnical investigation program to assess the stability of the 100-ft segment of the pipeline north of the existing retaining wall. Stability of the pipeline along other segments of the alignment would also need to be evaluated for final design of this alignment, because signs of instability were observed along segments of the alignment in the slopes east of the alignment.

3.2.2 Pipeline West of Polhemus Creek (IF-2)

In the on-site meetings that occurred between February 14 and 26, 1998, between the SFPUC and Golder Associates, an option was presented and discussed that involved construction of a new bypass pipeline on the west side of Polhemus Creek. For this alternative, the new pipeline would be passed under Polhemus Road, the existing 96-inch PCCP line, and a 72-inch RCP culvert that carries Polhemus Creek flows at the south end of the project. The pipe would then be raised in elevation above the influence of Polhemus Creek, then head south along the west bank of Polhemus Creek. In many locations along this alignment, no bench exists between the west bank of Polhemus Creek and the slope west of Polhemus Creek; much of the pipeline would be installed in the steep cross-slope of Polhemus Creek.

To evaluate Option IF-2, a geologic hazard reconnaissance was coupled with development of ground surface topography in early March 1998 [3]. A summary of the geologic reconnaissance, presented in Appendix E, was submitted to the SFPUC on March 18, 1998. Portions of that summary are presented in the discussion of site geology in Section 5.2.1.

Positive aspects of this alternative include:

- Less traffic disruption during construction than an alignment placed in Polhemus Road,
- Minimal impact to existing utilities,
- No negative impact to the repaired slope at the landslide site,
- A slightly higher expected level of survivability as evidenced by less historic and active sliding on the west slope as compared to the east slope.

Negative aspects include:

- High temporary and permanent biologic impact to Polhemus Creek and the vegetation on the slope west of Polhemus Creek as a result of the pipeline being installed along that slope,
- Poor access for construction equipment during construction,
- Poor access for maintenance equipment,
- Requirement of the installation and long-term operation of a booster pump, as topography dictates installation of the pipeline at elevations higher than the hydraulic gradeline,
- The steep slopes along much of the alignment have unacceptably low safety factors for a critical structure such as the proposed pipeline. Construction excavation will temporarily lower stability further and may induce slope failures. As a result, permanent retention systems will need to be constructed prior to construction along most of the alignment.
- Undercutting of these slopes could be inferred by upslope property owners as creating instability problems that could affect their properties.

se of the high biological impact, intense slope stabilization required to obtain a le pipe system, potential legal liability, and the capital and long-term operating associated with the need for a pump station, the UEB abandoned the concept of a s pipeline west of Polhemus Creek.

Pipeline West of Polhemus Creek at Landslide Area Only (IF-2a)

n IF-2a would be a combination of Options IF-1 and QF-2. In general, a new 84- steel pipeline would be constructed below the centerline of Polhemus Road except e it passes the repaired landslide. As in Option QF-2, the 84-inch steel pipe would to the west of Polhemus Creek between Stations 28+00 and 36+50. The pipe, e it crossed the creek at both crossings, would be structurally encased or pier orted. This option provides the same advantages of Option IF-1 with the added fit that the existing retaining wall at the landslide would not be impacted during ine construction. However, the environmental impacts to Polhemus Creek from the crossings were considered significant. When coupled with the same negative impacts of IF-2, this alternative was judged to be less favorable than Option IF-1 and was therefore eliminated from further evaluation.

3.2.4 Pipeline Adjacent to Retaining Wall (IF-3)

To limit construction costs and liability concerns associated with construction of tie-backs in the existing retaining wall north of Ascension Drive, a variation was proposed to the Polhemus Road centerline alignment. This variation focused on keeping the same alignment as that discussed in the previous section, except that the new pipeline would be constructed *above* the ground surface adjacent to the wall for the length of the wall, thus avoiding excavation along the wall and retention system constructed to repair the landslide. This alternative would require the installation and long-term operation of a pump station to maintain flow under normal operating conditions, and attendant capital and operating costs, and therefore was not considered for further study.

3.2.5 Pipeline Within Polhemus Creek (IF-4)

Working with Golder Associates, the UEB investigated the feasibility of the new bypass pipeline partially, or completely, in Polhemus Creek. This alignment would enter the creek just west of its beginning at the south end of the project and would exit the creek at Crystal Springs Road at the north end of the project. A detailed discussion of this study is included in Appendix F of this report.

The conceptual design for this option involved installation of both the proposed bypass pipeline and a water conveyance system that would carry Polhemus Creek flows in a double box culvert constructed in Polhemus Creek. The box culvert would have two 7 ft x 7 ft square cells aligned vertically, for a total culvert height of about 18 feet. The double box culvert configuration would minimize the amount of excavation needed for installation. The 84-inch steel bypass pipeline would be installed inside the upper culvert; the lower culvert would carry Polhemus Creek flows. The invert of the double box culvert would be at approximately the existing creek bottom grades.

Positive aspects of this alignment include:

- Less traffic disruption during construction than an alignment under Polhemus Road.
- Minimal impact to existing utilities.
- No negative impact to the repaired slope at the landslide site.
- A slightly higher level of survivability than alternatives closer to the east slope of the valley due to less historic and active sliding on the west slope when compared to the east slope.

Negative aspects include:

- High temporary and permanent biologic impact to Polhemus Creek and the vegetation surrounding the creek. In essence, Polhemus Creek would be routed through a culvert along the 4,000-ft long segment where the creek and pipeline would coincide and the creek would be filled.
- Construction would be limited to the dry season.
- High exposure to liability as construction of the box culvert would require undercutting of the slope to the west of the alignment, which could be inferred by upslope property owners as creating instability problems.

Permanent retention systems to increase stability of the slope west of Polhemus Creek along portions of the alignment would be required.

In particular, a critical segment of the alignment was identified to extend between about 800 feet to 1300 feet downstream from the Polhemus Road/Ascension Drive intersection based on steep topography. Preliminary stability analyses were performed by Golder for the existing slopes west of the creek in this area. Because actual subsurface information was not available, material property data developed for the landslide project had to be assumed for subsurface geologic conditions. Preliminary slope stability analyses indicated a static factor-of-safety against instability of only 1.1. A permanent method of slope retention would be required along this segment of the alignment prior to construction to stabilize the slopes so that pipeline stability was maintained at acceptable factors-of-safety. Other segments of the alignment, although not as critical as the section identified, would also require stabilization to provide a sufficient level of survivability to the new pipeline.

The biological impacts to Polhemus Creek were considered significant making this alignment's environmental clearance questionable. If approved, mitigation measures would be required that may add long delays to the project schedule. Slope stabilization requirements and the potential liability associated with this alignment ultimately caused the UEB to abandon it from further consideration.

3.3 Long-Term Options

Long-term options were those that could provide a high level of survivability when compared to short- or medium-term options, but generally increased cost and lengthened development schedule. These options focused on constructing the bypass at a sufficient depth in the Polhemus Creek Valley to minimize the concerns of impacts from slope instability, or on the use of an alignment located away from Polhemus Creek Valley.

3.3.1 El Camino Real Pipeline Alignment (LTF-1)

The El Camino Real alignment would provide overall transmission system redundancy to the existing 96-inch PCCP, and has been identified by the SFPUC in its long-term planning to improve transmission system reliability. The El Camino Real alignment could be a pipeline installed using open trench construction methods, or a tunnel boring machine driven through the El Camino Real alignment, for a distance of approximately 5 miles. The alignment would extend from the Redwood Valve Lot at the south to Hillsborough Valve Lot at the north, roughly following El Camino Real (Highway 82) for its entire length, passing through the cities of San Mateo and Burlingame, (Figure C-3).

In addition to providing redundancy to the transmission system, this alignment would provide a relatively high level of survivability, but could be costly. Estimated to cost $50 million, the development of this alternative would be at a far higher cost than the medium-term alternatives discussed above. In addition, right-of-way acquisition is anticipated to require several years. Current schedule estimates by the SFPUC indicate that this project is 15 to 20 years into the future. The SFPUC's present direction is that the existing 96-inch PCCP line should be bypassed, or replaced, sooner than that.

3.3.2 Crystal Springs Reservoir Pipeline Alignment (LTF-2)

The Crystal Springs Reservoir Pipeline would also provide transmission system redundancy by providing a "parallel" route to the CSBP. As shown on Figure C-3, this alternative would start at the downstream side of the Pulgas Pump Station, continue north along Skyline Boulevard, then easterly on Crystal Springs Road, and connect into the 78-inch diameter Sunset Supply Line, thereby connecting to the Hillsborough Valve Lot pipelines that draw water.

While this pipeline would also provide redundancy to the transmission system, as does the El Camino Real alignment discussed above, the survivability could be considerably less, becaue of its close proximity to the most active trace of the San Andreas Fault. As with the El Camino Real alternative, development costs of this alternative would be far higher than the medium-term alternatives discussed above. Right-of-way acquisition is also anticipated to require several years. With a completion date of this alternative scheduled 15 to 20 years into the future, the SFPUC's present direction is to bypass or replace the existing 96-inch PCCP line.

3.3.3 Polhemus Creek Valley Tunnel (LTF-3)

The Polhemus Creek Valley Tunnel (or Alternative 4 in this CER) would follow the general alignment of the existing CSBP. At an average depth of about 50 to 60 feet below the grade of Polhemus Road, this alternative would be installed below the influence of landslides in the walls of Polhemus Creek Valley. As a result, it would provide a higher level of survivability than would the medium-term options described in previous sections. While it would provide some redundancy to the existing 96-inch PCCP in Polhemus Road, it would not provide true transmission system redundancy. In addition to survivability, advantages include lower costs relative to the other long-term options; minimization of impact to existing utilities under Polhemus Road; and minimization of traffic impacts on Polhemus Road. Because the tunnel would be in

Polhemus Road, little if any new right-of-way acquisition would be required. Conversely, costs to construct this alternative would be significantly greater, and the development schedule significantly longer, than the medium-term alternatives presented. However, cost and schedule concerns were sufficiently low that the UEB considered this alternative worthy of further study. Reference 11 reported on the initial feasibility of the LTF-3 option.

Description	Advantages	Disadvantages	Conceptual Cost Estimate
QF-1 Repair the damaged PCCP: Replace approximately 60 feet of damaged PCCP at Sta. 32+50 (near the mid-length of the existing retaining wall on Polhemus Road).	• Lowest cost • Shortest duration of construction.	• Provides no protection against repeat of landslide damage; • Provides no redundancy in PCCP line under shutdown conditions.	$200,000
QF-2 Relocate a portion of the CSBP: Relocate a portion of the PCCP line, from Sta. 28+00 to Sta. 36+50. At Sta. 28+00, the new steel pipe will connect to the existing PCCP line and cross to the west side of the creek (via structural encasement underneath creek, or via supports spanning over the creek) where it will run along the west bank of the creek to a point past the existing landslide area. It will then cross the creek likewise again and connect back to the PCCP line near Sta. 36+50.	• Alignment is located away from landslide area. • Connections to existing PCCP may be made during scheduled shutdowns.	• Environmental impacts are significant due to two creek crossings; • West bank has steep, vegetated slopes; • Access to maintenance is difficult; • Does not provide redundancy to PCCP during shutdown for routine maintenance.	$1.8 million
IF-1 Install steel pipe at the centerline of the road: Construct a new steel pipeline parallel to the east of the existing CSBP beginning in the Polhemus Valve Lot and ending on Crystal Springs Road. The new 84-inch pipeline will connect to the system in Polhemus Valve Lot and run 15 feet center-to-center to the existing 96-inch PCCP along Polhemus Road. After passing the landslide area, it will split into two branches: a 60-inch Crystal Springs Pipeline No. 2, and the 79-inch Sunset Supply Pipeline.	• Provides local redundancy to pipeline operations for shutdowns, • Accessibility to routine maintenance and repairs, • Constructability, • Relatively lesser environmental impacts.	• Traffic maintenance of Polhemus Road during construction, • Nearness to existing slide and exposure to future similar landslides.	$8 million (without slope stabilization)
IF-2 Install steel pipe at the center of the road, bypass to the west of the creek and back to the road: A new steel pipeline would be constructed in the road, east of the existing 96-inch PCCP. South of the landslide area, the pipe will cross to the west banks of the creek north of the landslide area, will cross the creak again and branch into a 60-inch and 78-inch pipeline.	• Provides local redundancy • Landslide area will not impact the pipe.	• Crosses the 96-inch PCCP, • Environmental impacts.	$10 million
IF-2a Install steel pipe to the west of Polhemus Creek: A new steel pipeline would be constructed west of the creek for the entire length of the alignment. The new 84-inch pipe would connect to the system at the Polhemus Valve Lot and tie into the Crystal Springs Pipeline No. 2 and Sunset Supply Pipeline on Crystal Springs Road.	• Provides local redundancy to pipeline operations for shutdowns, • Less traffic disruption during construction, • Minimal impact to existing utilities, • No negative impact to repaired slope at landslide, • Excavation depth is relatively shallow.	• High environmental impact to creek, • Poor construction access, • Poor access for long-term maintenance, • Stability concerns in slope west of pipeline could require significant stabilization, • May require booster pump.	Not estimated

Table C-1 Summary of Preliminary
Conceptual Design Alternatives
To Replace Existing 96-inch PCCP
Sheet 1 of

IF-3 Install steel pipe at the center of the road, then transition to an above-ground pipe next to the retaining wall, then back underground.	• Provides local redundancy for shutdown of existing PCCP, • Avoids disturbing retaining wall structure.	• Remains exposed to future landslides in the area, • Traffic maintenance of Polhemus Road during construction. • Requires booster pump station.	$12.4 million
IF-4 Install steel pipe in Polhemus Creek: A new steel pipeline would be constructed in Polhemus Creek for the entire length of the alignment. The new 84-inch pipe would connect to the system at the Polhemus Valve Lot and tie into the Crystal Springs Pipeline No. 2 and the Sunset Supply Pipeline on Crystal Springs Road. The steel pipe would be constructed in the upper cell of a 7 feet square double box culvert placed vertically. The lower cell would carry Polhemus Creek flows.	• Less traffic disruption during construction, • Minimal impact to existing utilities, • No negative impact to repaired slope at landslide.	• High temporary and permanent environmental impact to creek, • Poor construction access, • Poor access for long-term maintenance, • Stability concerns in slope west of pipeline could require significant stabilization.	Not estimated
LTF-1 Connect BDP Nos. 1 and 2 at Redwood City Valve Lot to either Bellevue and Pepper Valve Lot of Capuchino Valve Lot in Millbrae along El Camino Real alignment.	• Provides system redundancy, such as in case of damage and shutdown of existing PCCP; • Located away from landslide area.	• Additional time for detail planning studies, • Probably requires full EIR, • Traffic impacts along El Camino Real, • Utility relocations required, • Interagency coordination and agreements.	$50 million
LTF-2 Pulgas Tunnel via Canada Road to Skyline Boulevard to Crystal Springs Road.	• Provides system redundancy, such as in case of damage and shutdown of existing PCCP; • Located away from landslide area.	• Additional time for detail planning and feasibility studies, • Crossing of San Andreas fault, • Environmental impacts on undeveloped land require EIR.	$30 million
LTF-3 Tunnel under Polhemus Road: A tunnel would be constructed in Polhemus Valley for the entire length of the alignment to bypass the existing 96-inch PCCP. In general, the tunnel would be driven beneath Polhemus Road at a depth of about 50 to 60 feet to be beyond the influence of landsliding. (Alignment approximates IF-1).	• Less traffic disruption during construction, • Minimal impact to existing utilities, • No negative impact to repaired slope at landslide, • Minimal environmental impact, • Higher reliability as tunnel will be below influence of surficial natural hazards, such as landslides and flooding, • Lower costs compared to other long-term options.	• High development costs relative to medium-term options, • Longer development schedule compared to medium-term options.	$25 million

Table C-1 Summary of Preliminary Conceptual Design Alternatives To Replace Existing 96-inch PCCP
Sheet 2 of 2

Appendix D

History of Polhemus Road Landslide and Slope Repairs

APPENDIX D

HISTORY OF POLHEMUS ROAD LANDSLIDE AND SLOPE REPAIRS[1]

1. Introduction

The City and County of San Francisco Public Utilities Commission (SFPUC, or CCSF) owns and operates the 96-inch diameter Crystal Springs Bypass Pipeline (CSBP) as part of their water system that conveys water from Hetch Hetchy Reservoir in the Sierra Nevada foothills to end users in San Francisco. The 4,250-ft long pipeline is located beneath Polhemus Road in San Mateo County southeast of the City of City of San Mateo between State Highway 92 and Crystal Springs Road. The 96-inch diameter pipeline was constructed in the 1960's of Prestressed Concrete Cylinder Pipe (PCCP). Figure 1 shows the approximate location of the pipeline.

A large landslide mobilized in 1997 and 1998 along Polhemus Road in the slope above the existing 96-inch CSBP, threatening the integrity of the pipeline and requiring emergency response by the SFPUC. The slide also directed attention to the damage vulnerability of the 96-inch PCCP line in a valley that has had past landslides and in which signs of active slope movements have been identified.

The following discussion summarizes the actions implemented to protect the 96-inch PCCP line and stabilize the slope above the pipeline after the landslide had mobilized.

2. Landslide

In the winter of early 1996, distress cracks were observed near the crest of a steep,150-ft high slope that extended between Polhemus Road at its base and Rainbow Drive at its crest. The distress occurred at a location centered about 1,000 feet north of the intersection of Ascension Drive with north to south trending Polhemus Road. The distress was characterized by cracks extending about 250 feet in length through the rear yard areas of four residences. The property owners of the four residences retained a geotechnical consultant to perform a geologic investigation and to provide mitigation recommendations to stabilize the rear yard areas. Mitigation ultimately included construction of a tied-back soldier pile retaining wall spanning the rear yard area of the four residences.

In mid-January 1997, an approximately 350-ft length of the slope between Polhemus Road and Rainbow Drive mobilized. The landslide destroyed the new retaining wall at the crest, destroyed one of the four residences, threatened the other three residences, and covered the paved Polhemus Road below (which is maintained by San Mateo County). An air photograph of the slide, taken in February 1998, is presented as Figure 2.

The Water Supply and Treatment Division (WS&TD, or "Water Department") of the SFPUC operates the 96-inch diameter bell-and-spigot joint RCCP under Polhemus Road within the slide area. The 96-inch diameter pipe extends from a valve vault located about 2,300 feet south of the intersection of Ascension Drive with Polhemus Road at its south end to Crystal Springs Road at its north end. With the exception of a crossing of San

---

[1] Prepared by Russ E. Browne, P.E., Golder Associates, May, 2000.

Mateo Creek near its north end, the pipeline, built in the 1960's, lies under the west shoulder of Polhemus Road. The pipeline links the Pulgas Tunnel to the south with the Sunset Supply Pipeline and the Crystal Springs No. 2 Pipeline to the north. The CSBP is operated with about 10 feet of total head (fluid pressure) with the crown of the pipe located about 3 to 5 feet below the surface of Polhemus Road.

With the slide covering Polhemus Road, the Water Department immediately became concerned that additional vertical loading of the CSBP, below the road, could cause structural damage to the pipe.

Litigation ensued between the property owners at the top of the slide and San Mateo County, who operate and maintain Polhemus Road at the base of the slide. The slide, instrumented with piezometers and inclinometers after the initial movements, stopped moving after cessation of winter storms. Throughout the spring, summer, and fall of 1997, little was accomplished in the way of remediation of the slide. A temporary one-lane road was routed over the toe of the slide to maintain traffic flow on Polhemus Road.

In late January 1998, winter storms reinitiated the slide. The thickness of slide debris increased over the top of the 96-inch PCCP line to a limit that the Water Department and the Utilities Engineering Bureau (UEB) of the SFPUC felt may be causing structural damage to the waterline. The plan limits of the landslide at the end of February 1998 are shown on Figure 2. After a Cal Water pipeline beneath Polhemus Road ruptured, and Water Department staff observed water flowing from the toe of the slide that was thought to be from a source other than the Cal Water pipeline, the Water Department and UEB became concerned that the observed water could be flowing from a possible leak in the 96-inch PCCP line that may have resulted from overstressing of the pipe. Based on this concern, two parallel actions were initiated: first, a more detailed investigation to determine the extent of damage and impact to the pipe was to be conducted with the goal of reducing pipe impact; and second, the UEB was to begin design of a new pipeline that would provide redundancy to the existing pipeline in the case of failure of the existing pipeline.

3. Emergency Response to Address Slide Impact to Pipe

Responding to a request by the SFPUC, the UEB, formed an engineering investigative team through the consultant joint venture of GKO & Associates and Golder Associates Inc. (GKO/Golder) to assess slide impact to the 96-inch PCCP line. Representatives of the SFPUC and the engineering team at the site on February 2 and 3, 1998. GKO/Golder and the SFPUC discussed the history of the slide and the potential for vertical loading damage to the 96-inch PCCP line. In this meeting, Golder expressed additional concern of a potential for damage to the waterline that could result in the added weight of the toe of the slide causing instability of the east slope of Polhemus Creek, which in turn could create horizontal deformation of the waterline.

During these site meetings, Golder performed a preliminary geologic reconnaissance of the slide and identified signs of recent movement in the slide mass that included: fresh tension cracks and headscarps near the head of the slide; a fresh lateral scarp at the north end of the slide; pressure ridges and over-steepened topography at the toe of the slide; and buckling and uplifting of asphalt in the temporary paved Polhemus Road bypass that crossed the toe of the slide. After a thorough inspection of the eastern slope of Polhemus Creek (located west of the 96-inch PCCP line and the toe of the slide), Golder observed that there was no visible evidence of recent movement of the slope into the creek, which might be characteristic of a deeper slide plane or a sympathetic slide plane, that could affect the integrity of the 96-inch PCCP line. However, the SFPUC and Golder agreed that an emergency action and monitoring program be initiated immediately.

The first part of the program involved notification by the CCSF to San Mateo County (County) so that the County could consider an immediate road closure for public safety. It was the County's decision to keep the temporary bypass of Polhemus Road open under heightened monitoring.

At the request of the UEB, Golder immediately initiated development of an investigation and monitoring plan to more fully evaluate the effect of renewed movement of the slide on the 96-inch PCCP line and to prepare solutions to prevent damage to the waterline should the evaluation indicate slide-induced waterline movement. The plan involved data collection and review, geologic reconnaissance and investigation, installation and monitoring of field instrumentation, and development of existing detailed ground surface topography in the slide area.

The field investigation ensued, which included conducting a detailed geologic reconnaissance of the slide to map surface expressions of the landslide to supplement available subsurface information. Starting on February 8, 1998 and working around the clock until completion, a boring (Boring B-1) was drilled through the slide into the underlying Franciscan formation. Located at approximately Station 109+50 (referencing final pipeline design stationing) and about 38 feet west of the 96-inch PCCP line (between the waterline and Polhemus Creek), Boring B-1 was fitted on February 9, 1998 with an inclinometer, denoted PR-1. The depth and location of Inclinometer PR-1 was selected to determine if there was a deeper failure surface (deeper than that of the observed surface sliding over the top of Polhemus Road) that might be impacting the 96-inch PCCP line. After an initial baseline reading, the first recorded reading of Inclinometer PR-1 on February 13, 1998 indicated a horizontal deformation rate of 0.15 inches per day at a depth of 14 feet. This depth corresponded to an elevation nearly the same as the invert (base) of the 96-inch PCCP line. Monitoring of PR-1 was continued until its ultimate failure on March 9, 1998 as a result of continuous and focused shearing at a depth of 14 feet. During the monitoring period of Inclinometer PR-1, lateral deformation (shearing) at the depth of 14 feet remained relatively constant at a rate of 0.15 inches per day, resulting in a total measured deformation of 3-1/2 inches before the inclinometer casing was sheared off.

Concurrent with the inclinometer readings, surface monitoring devices were installed near Inclinometer PR-1 to track slide movements. Surface monitoring was initiated on February 13, 1998.

Because of the high rate of deformation observed in Inclinometer PR-1 at an elevation corresponding to that of the invert of the pipe, it was concluded that there was a real risk of damage to the 96-inch PCCP line from the slide and that emergency measures were warranted to arrest continued movement of the soil around the pipe. Between February 14, 1998 and February 26, 1998, GKO/Golder prepared and presented emergency mitigation options to the UEB in a series of meetings. Four final options were presented to the UEB in a letter dated February 26, 1998. By this time, a preferred option had been selected by the UEB. The preferred option involved stabilizing the deeper observed failure surface delineated by Inclinometer PR-1 through construction of a buttress or fill in Polhemus Creek. The fill option incorporated a bypass culvert(s) to convey storm flows through the fill. The intent of this option was not to stabilize the entire landslide mass but only to protect the 96-inch PCCP line against movement.

Again, working under an emergency response-driven accelerated schedule, GKO/Golder completed a draft design of the Polhemus Creek buttress on March 5, 1998. After incorporating the results of discussions in meetings with the SFPUC, the final draft of the buttress design was submitted to the SFPUC on March 16, 1998. In essence, the design involved filling Polhemus Creek with a rock fill to arrest deep movements that could affect the 96-inch PCCP line. Full hydrologic and hydraulic analyses were conducted for GKO/Golder's design of bypass culverts in the buttress to allow the passage of most of the estimated 100-year peak flow of 573 cubic feet per second. Additional flow capacity to convey the balance of the 100-year peak flow was ultimately provided by an open channel graded into the surface of the buttress fill.

The GKO/Golder team evaluated options for hydraulic passage of the storm flows through the buttress. The selected option consisted of the installation of three 30-inch diameter polyethylene drainage pipes through the fill. The final draft documents submitted to the SFPUC on March 16, 1998 included detailed construction drawings and technical specifications to a sufficient level for construction.

The buttress was installed by the CCSF in March 1998. Headwall construction for the culverts was continued in April 1998.

Because of the failure on March 9, 1998 of Inclinometer PR-1, additional inclinometers were installed by Golder to monitor the performance of the temporary buttress. Inclinometer PR-2, completed on April 14, 1998, was installed in a boring drilled at approximately Station 109+65 about 16 feet east of the centerline of the 96-inch PCCP line. Inclinometer PR-3, completed on April 18, 1998, was installed adjacent to the disabled Inclinometer PR-1. In addition, two piezometric standpipes were installed to measure the presence of subsurface water in the landslide mass and landslide foundation. Piezometer P-1 was installed adjacent to Inclinometer PR-2 and Piezometer P-2 was installed adjacent to Inclinometer PR-3. The piezometers and inclinometers were installed with collar elevations similar to road surface elevations.

The readings from Inclinometer PR-3 (replacement for PR-1) confirmed that the temporary buttress did in fact effectively stop the earth movements that were measured in Inclinometer PR-1 at a depth corresponding to the base of the 96-inch PCCP line. The high rate of strain across the Inclinometer PR-2 (at road surface elevations) sheared the inclinometer casing within days of its installation. Inclinometer PR-3 remains accessible and operational.

Although deep movement that could cause lateral deflection of the waterline had been arrested by the buttress fill, the main slide remained mobile. Interpretation of surface and instrumentation monitoring of the slide for a period of nearly one month revealed that the failure mode of the slide mass as it passed over the 96-inch PCCP line was translational; it was confirmed that the mass was sliding over the Polhemus Road surface.

As an additional measure in the emergency response mitigation plan recommended by Golder to protect the waterline, temporary surface water controls were constructed on the surface of the slide which included:

- covering a portion of the landslide with plastic sheeting to reduce infiltration;
- minor grading of the landslide mass to promote runoff and reduce infiltration;
- constructing interceptor trenches along the crest of the landslide headscarp;
- intercepting runoff from the residences upgradient of the slide; and
- tight-lining surface water away from the slope to exit into Polhemus Creek at the toe of the slope.

The recommendations were presented by Golder in March 20, 1998 letter.

4. Preliminary Permanent Remediation Design

On May 14, 1998, GKO/Golder, working together with the UEB, initiated development of alternatives to remediate the entire slide mass out of concern for the steadily increasing overburden loads on the 96-inch PCCP line. After completion of a preliminary design of the two most promising alternatives, a construction cost estimate was prepared to construct the two options. The options along with estimated capital costs were presented to the SFPUC in a letter dated May 22, 1998. A final copy of *the Preliminary Study – Alternatives for Remediation of Landslide Hazards* was submitted to the UEB on May 29, 1998 (Golder 1998, Reference 7).

On May 29, 1998, GKO/Golder presented the UEB, at their request, with a proposal to perform final field characterization of the landslide and to perform final design of each of the two remedial options presented by GKO/Golder. The UEB was to select a preferred option and retain Golder to perform final geologic and geotechnical characterization and prepare a final design of that option.

5. Final Design and Construction

In late May 1998, the SFPUC reached an agreement with San Mateo County that placed design and construction responsibility of the permanent remediation of the landslide with a joint venture of Soil Engineering Services and Soil Construction Services (referred to as the Joint Contractor Team or JCT). The remediation contractor retained Cotton, Shires & Associates, Inc. (CSA) to perform detailed geotechnical and geological characterization of the slide and to prepare design documents for remedial construction of the slide. The agreement required that the SFPUC and their consultant(s) perform technical review of the contractor's design and construction submittals during the remediation. Golder was retained by the UEB to provide technical review of the CSA design with respect to protection of the 96-inch PCCP line. In addition, Golder was asked to continue to monitor active inclinometers and surface monitoring devices adjacent to the 96-inch PCCP line that had been previously installed by Golder. The terms of the agreement were discussed in a meeting at the San Mateo County offices on June 9, 1998.

Cotton, Shires & Associates, Inc. immediately initiated monitoring and geologic characterization of the slide. They drilled, logged, and installed inclinometers in seven borings the week of June 8 through 12, 1998. Four large diameter holes were drilled between the dates of June 17 and June 24, 1998 to allow down-hole logging of the slide and underlying Franciscan Formation. Geotechnical laboratory testing followed.

A conceptual remediation design was presented by CSA on July 9, 1998, along with a geologic characterization of the landslide, as depicted in geologic sections. The original design concept established three phases of construction. Phase I involved stabilizing the headscarp of the slide through installation of shear pins (drilled piers fitted with wide-flange beams). Phase II included removing the landslide debris at the toe of the slide and installing a cantilever soldier pile wall. The 2-foot diameter drilled soldier piles that provided the structure for the wall were spaced at 6 feet on center and extended 40 to 45 feet below the surface of Polhemus Road. Steel beams, held in place in the piles with concrete, extended upward from the soldier piles 25 feet to form a 25 feet high retaining wall. Wood lagging was installed between the vertical steel beams. As fill was brought up behind the wall, the soldier piles were tied back with a single row of tie rods to a continuous concrete deadman constructed behind the wall.

Phase III involved excavating the remainder of the landslide, excavating a keyway at the toe of the slide adjacent to the Phase II soldier pipes, and rebuilding the slope as a compacted fill at a 2H:1V (horizontal to vertical) slope projecting up from the top of the retaining wall.

Construction of the Phase I shear pins started July 29, 1998 and was completed by August 15, 1998. Drilling of the Phase II soldier piles was completed in two stages; October 22, 1998 to November 6, 1998 and on May 26, 1999. Excavation and backfilling of the new slope continued through the winter of 1998/1999 with occasional delays due to weather. Wet weather completely stopped the filling operation from late February 1999 to late April 1999. Filling was completed on June 6, 1999 (CSA 1999).

Appendix E

Field Reconnaissance for Diversion of Polhemus Road Water Transmission Pipeline

Appendix F

Polhemus Creek Alignment Option Study

DATE: February 27, 2000 GOLDER REF: 983-7004, L00-316
TO: Russ Browne, Golder
Project File
FROM: Barry MacDonnell

RE: **POLHEMUS CREEK ALIGNMENT OPTION STUDY**
84-INCH POLHEMUS ROAD BY-PASS PIPELINE

INTRODUCTION AND PROJECT DESCRIPTION

The Utilities Engineering Bureau (UEB) of the San Francisco Public Utilities Commission (SFPUC) is considering an alternative alignment to their proposed 84-inch diameter steel water transmission pipeline that will replace the existing 96-inch diameter Crystal Springs Bypass Pipeline (CSBP) in San Mateo County. The 96-inch CSBP is part of the SFPUC's water system that conveys water from Hetch Hetchy Reservoir in the Sierra Nevada Foothills to end users in San Francisco. The existing 4,250 feet long 96-inch CSBP, constructed in the 1960's of Prestressed Concrete Cylinder Pipe (PCCP), is located beneath the west shoulder of Polhemus Road. The alignment is shown on Figure 1.

The UEB has completed design (to the 85% level) of the new 84-inch diameter steel pipe to replace the existing 96-inch PCCP line. Under the current design, the new pipeline will be located east of the existing pipeline under the center of Polhemus Road. After installation of the 84-inch steel pipeline, the existing 96-inch PCCP line will not be removed, but will remain in service at a reduced flow capacity and as a temporary bypass to the new 84-inch steel pipeline during scheduled inspections and maintenance of the new pipeline.

For this study, Golder, working together with the UEB, investigated the feasibility of alternative alignment for the new 84-inch steel pipeline that would be located partially, completely, in Polhemus Creek instead of under Polhemus Road, as currently designed. T alignment would be approximately 4,250 feet long and would enter the creek at its south e approximately 2,500 feet south of the intersection of Polhemus Road and Ascension Drive a exit the creek at its north end near Crystal Springs Road. The discussion presented belo references the stationing developed for the currently-designed new 84-inch steel pipeline presented on drawings prepared by the UEB entitled *84 inch Crystal Springs Bypass Pipelin Polhemus Road, San Mateo County*, dated September 10, 1999.

The potential benefits of installation of the pipeline in the creek that prompted this study were:

1. Less traffic disruption on Polhemus Road during pipeline construction than the currently-designed pipeline;
2. Less impact to existing underground utilities during construction than the currently-designed pipeline;
3. Less impact to potentially unstable slopes east of Polhemus Road along segments of the new pipeline during trenching for new pipeline construction; and
4. No impact to the existing retaining wall and repaired slope at the site of the repaired Polhemus Road Landslide. The Polhemus Road Landslide, located between about Stations 29+00 and 32+50, was repaired in 1998 and 1999 through the construction of a 350 feet long soldier pile cantilevered retaining wall and reconstruction of the slope behind the 25 feet high retaining wall. The soldier pile wall extends down about 40 to 45 feet below the surface of Polhemus Road. Previous studies indicated that the trenching for the new 84-inch steel pipeline (alignment centered near the centerline of Polhemus Road) would relieve passive resistance to the soldier pile wall, lowering the factor-of-safety against stability of the repaired slope. Construction of the pipeline in Polhemus Creek would eliminate the need for

- Grades of 1-1/2 to 2 percent between Stations 0+00 and 30+00;
- Grades of 5 to 6 percent between Stations 30+00 and 37+00; and
- Grades of 7 to 9 percent from Station 37+00 to the confluence of Polhemus Creek with San Mateo Creek at about Station 42+50.

Local grades are as steep as 12 percent.

PROPOSED ALIGNMENT AND CONSTRUCTION

The conceptual design for the Polhemus Creek alignment centers on conveying both Polhemus Creek flows and pipeline flows through a double box culvert constructed in Polhemus Creek. The box culvert would have two 7-feet wide by 7-feet high square cells aligned vertically. The total height of the culvert (outside dimensions) would be about 18 feet. Construction of the double box culvert in a vertical configuration would minimize the amount of excavation needed for installation. The 84-inch diameter by-pass pipeline would be installed inside the upper culvert; the lower culvert would carry Polhemus Creek flows.

For this conceptual study, Golder verified that the lower culvert could convey the estimated 100-year storm event discharge of 1,200 cubic feet per second. The existing 72-inch and 78-inch RCP's, and the 5 feet wide by 3.5 feet high concrete box culvert described above, are not sufficiently large to convey flows from the 100-year storm event and would therefore require replacement with the double box culvert during pipeline construction.

At the southern end of the alignment, the currently designed 84-inch steel pipeline manifolds from the existing 96-inch pipe to the north. The Polhemus Creek alternative would require the 84-inch pipe to manifold from the existing 96-inch pipe to the south to prevent the need to cross the existing 96-inch pipe where the 84-inch pipe crosses Polhemus Road and enters Polhemus Creek.

The double box culvert that would carry the new pipeline would be constructed in Polhemus Creek. To limit impact on the creek and reduce the risk of storm washouts, construction would be sequenced to minimize the length of construction in and along Polhemus Creek at any one time. In addition, construction would need to progress in the middle- to late-summer when creek flows were minimal and could be controlled by temporary damming upstream of construction and pumping creek flows through temporary piping around the active construction zone.

Once temporary water controls were in place, all vegetation would be removed from the base and slopes of the creek, as well as from between the creek and Polhemus Road (for construction staging and access). The base of the creek would be widened with earth moving equipment to a temporary width of about 15 feet sufficient for culvert construction and backfilling operations. The box culvert would be constructed at grades that mimicked the existing channel grades and would installed with a minimal amount of bends to minimize operational friction losses in the 84-inch pipe and for constructability of the box culvert/pipeline system. Pipeline installation would be concurrent with box culvert installation, requiring close coordination between the culvert- and pipe-laying operations.

A significant grading effort would be required to straighten the gently meandering stream channel to minimize grade breaks and pipe bends in the box culvert (and pipeline). In addition, expanding the base of the creek from the current width of 5 to 10 feet to a width of at least 15 feet would also require a significant earthmoving effort.

The required grading in the creek would impact slope stability of the relatively steep slope west of Polhemus Creek. In particular, critical segments of the alignment were identified:

- Stations 7+00 to 9+00,
- Stations 29+00 to 33+50 (adjacent to the existing retaining wall and a 100 feet length north of the wall), and
- Stations 33+50 to 40+00.

The slopes west of Polhemus Creek along these segments of the alignment were characterized as having relatively steep slopes (locally 1H:1V to 3H:1V). Excavation for the box culvert would create steepening at the base of these slopes, resulting in a temporary lowering of static stability. At the time this study was prepared, subsurface geologic and groundwater conditions were not known. As a result, detailed stability analyses could not be performed to predict the effect of temporary excavations. Golder did, however, perform conceptual-level stability analyses using actual slope geometries (where available from topographic maps), soil strength properties developed during repair of the landslide, and assumed subsurface water levels. Two stability concerns were reviewed; temporary slope stability during construction and long-term slope stability and the potential for damage to the box culvert and pipeline from a landslide after installation.

Our conceptual-level stability analyses indicated that there is a high probability that temporary excavations along these critical segments of the alignment would drop the factor-of-safety to less than acceptable safety standards, possible creating unstable conditions that could affect the slopes west of the creek, unless excavations were rigidly braced. In addition, our analyses indicated that the pre-construction slopes west of Polhemus Creek along portions of these critical segments already have factors-of-safety less than generally-accepted industry standards (1.4 to 1.5). This doesn't infer that the slopes are not currently statically stable, but rather suggests that the factors-of-safety against failure surfaces that could damage the proposed culvert and pipeline may not meet industry standards. Construction of the box culvert would have little effect, if any, on increasing long-term factors-of-safety of these slopes above the culvert/pipeline system. As a result, permanent slope strengthening or stabilizing measures would need to be implemented in the slopes west of Polhemus Creek in these critical areas prior to construction to create an acceptable factor-of-safety along a failure surface that could impact the culvert/pipeline system. Depending on actual subsurface geology and groundwater conditions, such mitigation measures could include: installation of hydro-augers to drain the slopes, installation of permanent tie-back anchors, soil nailing, or retaining wall construction.

ALIGNMENT ADVANTAGES AND DISADVANTAGES

The benefits of the Polhemus Creek alignment were listed above and include:

1. Less traffic disruption on Polhemus Road during pipeline construction than the currently-designed pipeline;
2. Less impact to existing underground utilities during construction than the currently-designed pipeline;
3. Less impact to potentially unstable slopes east of Polhemus Road along segments of the new pipeline during trenching for new pipeline construction; and
4. No impact to the existing retaining wall and repaired slope at the site of the repaired Polhemus Road Landslide.

The following summarizes the negative aspects of development of the pipeline in Polhemus Creek.

1. Construction would be limited to the dry summer months to reduce temporary dewatering requirements. Even with summer construction, there would still be a significant risk of storm events, which could cause delays in the construction schedule, damage to installed facilities,

and uncontrolled sediment runoff from stripped and/or graded areas. Construction along this alignment would require development of a detailed work plan that focused on surface water control and minimization of erosion and excess sediment loading of downstream creeks. In addition, an emergency response plan would need to be developed that addressed actions that would be implemented in the case of an impending storm that could overwhelm temporary surface water controls.

2. Development along this alignment would have a high temporary and permanent biological impact to Polhemus Creek and the vegetation surrounding the creek. Temporarily, all vegetation and wildlife would be eliminated. Restoration of vegetation would take years. Some wildlife habitat, all wetlands, and all aquatic plant species would be permanently lost and would likely require mitigation measures such as purchasing additional habitat near the site. The potential for erosion and downstream siltation during construction would be very high.
3. There would be permanent degradation of the visual character of the creek throughout the length of the alignment as the open creek would be eliminated.
4. Permanent retention systems will likely be required in the slopes west of the creek along segments of the alignment to ensure safety of the pipeline against landslide-induced damage at acceptable safety standards.
5. Compared with the existing proposed alignment under Polhemus Road, the development schedule could be dramatically lengthened if the Polhemus Creek alternative is selected. There is a possibility that this alignment could be completely blocked against development by regulatory agencies, including the California Department of Fish and Game, the Regional Water Quality Control Board, the U.S Fish and Wildlife Service, or the U.S. Army Corps of Engineers. If mitigating measures are proposed and approved and the project is allowed to proceed, schedule impacts would remain from:

- the need to perform a full geotechnical investigation along the alignment, to perform detailed stability analyses to help ensure protection of valley slopes during construction, to properly design retention systems, and to prepare design and construction drawings for the culvert and pipeline;
- the likely requirement of preparation of an EIR;
- the time required to develop mitigating actions or to purchase nearby property to mitigate project biological impacts; and
- the potential for constructing the box culvert and pipeline in two construction seasons if construction occurs during a wet year.

Appendix G

Geology and Geologic Hazard Reconnaissance of the Slope East of Polhemus Road

DATE: April 28, 2000 GOLDER REF: 003-7164, L00-317
TO: BARRY MACDONNELL
RUSS BROWNE
PROJECT FILE 003-7164
FROM: Alan Pace

RE: GEOLOGY AND GEOLOGIC HAZARD RECONNAISSANCE OF THE SLOPE EAST OF POLHEMUS ROAD

A geologic reconnaissance of the area east of Polhemus Road was performed on April 19th and 20th 2000, by Golder Associates Inc. (Golder). The objective of the reconnaissance was to identify the general geologic setting of the project and evaluate the presence of landslides which may affect the proposed development of a pipeline or tunnel under Polhemus Road to bypass the existing 96-inch Crystal Springs Bypass Pipeline. This study supplements a geologic reconnaissance performed by Golder in 1998 which was performed to evaluate a potential alignment of a new pipeline to the west side of Polhemus Creek. In that study, Golder identified shallow landslide debris flows west of Polhemus Creek.

The reconnaissance included the following activities:

- The evaluation of approximately 4,250 feet of potential pipeline route along the east side of Polhemus Creek, extending from near the confluence of San Mateo and Polhemus Creeks (on the north), to the construction staging area 4,250 feet to the south along Polhemus Road;
- The evaluation of stereographic aerial photographs: 7-29-46, GS-CP, scale 1:23,600; 12-30-69, AV-933-09, scale 1:12,000; 9-7-95, AV 4916 309, scale 1:12,000; 8-5-97, SMT AV 5434 309, scale 1:12,000; 2-26-98, 986452, scale 1"=200', and

- The evaluation of geologic conditions and potential geologic hazards (e.g., landslides, slumps, rockfalls, rockslides, lateral creek erosion, etc.) along the study route.

The topography in the area varies from the relatively gentle slope of Polhemus Creek, which flows to the northwest, to the near vertical bedrock slopes east of Polhemus Road between stations 33+00 and 39+00. Slopes west of Polhemus Creek commonly dip on the order of about 45°. Elevations vary from about 225 feet near the confluence of San Mateo and Polhemus Creeks to about 550 feet along the edge of the ridgeline located west of Polhemus Creek.

General geologic units (Brabb and Pampeyan, 1983) observed along the proposed alignment include:

Quaternary Age, Unconsolidated, Young Alluvial Deposits – located along and adjacent to the creek bottom and composed of sand, silt, clay, gravel, and boulders.

Quaternary Age, Unconsolidated, Colluvial Deposits – located on slopes and composed of loose to firm, friable, unsorted sand, silt, clay, rock debris, and organic material.

Quaternary Age Landslide Deposits – consisting of unconsolidated colluvial and bedrock deposits.

Jurassic and Cretaceous Age Bedrock of the Franciscan Complex - consisting of mostly sandstone, siltstone, greenstone, greywacke, and shale (which is commonly sheared), with a small amount of serpentinite in the northern portion of the reconnaissance area.

During the field reconnaissance, potential geologic hazards were evaluated for the slopes above and banks adjacent to Polhemus Road by looking for indicators of recent or past slope movement, erosion, sedimentation, and unstable conditions (e.g., landslide head scarps and debris toes, fallen trees, erosional rills, slumping, rockslides, rockfalls, debris flows, lateral creek erosion, unvegetated bedrock and sediment, etc.). An additional potential geologic hazard includes flooding which may result from the damming of a drainage by a slope failure (e.g., landslide, slump, rockfall, etc.). The subsequent failure of the dam could result in an uncontrolled flood accompanied by excessive erosion and damage downstream.

The observed geologic features are described below:

- **Station 0+00 to Station 12+75** – Between these stations, the slopes were characterized as relatively steep and moderately to heavily vegetated with trees and shrubs. Based on our review of the 1946 aerial photographs, the slope above the road in this area was occupied by an ancient landslide but does not appear to be currently active. Shallow instability in the form of surface slumps was noted during the reconnaissance. However, rockfalls should be expected. Moderately weathered rock (sandstone) was exposed at the base of the slope in most of this area with occasional outcropping boulders of lightly-weathered shale. The sandstone was characterized as medium grained, moderately cemented and thickly bedded to massive.
- **Stations 12+75 to 23+50** - The slope above the road along this segment of the alignment had an inclination of approximately 2 ½:1 (H:V) and was vegetated primarily with grasses. Trees and shrubs occupied minor portions of the slope with phreatophyte reeds and sawgrass occupying the slope near Station 17+75. The majority of this slope was mantled with

small debris flow and soil slips resulting in hummocky topographic expression. Rock units were not observed at the surface in this reach with the exception of a boulder "nest" near Station 19+50 which was also moderately vegetated with large trees and shrubs. The rocks in this boulder nest were characterized as Greenstone of the Fransican Formation and were hard to very hard and similar to the hard rock unit encountered in the keyway excavation for the slope repair to the north (Cotton and Shires, 1999). The upper portion of this slope beneath the residential development showed displacement of the topography which had exposed the foundation of four of the homes situated on the hillside above. The 1946 aerial photos showed geomorphology indicative of large scale landsliding. This evidence included hummocky topography, arcuate backscarp areas, and apparent deflection of the stream at the toe of the slide. It should be noted that distress in the road crossing the toe of this suspected landslide was not noted during this reconnaissance indicating larger and older landslides may have pre-dated construction of Polhemus Road.

- **Stations 23+50 to 28 +00** - The slopes between these stations were gentle and also displayed characteristics of older landslides. Rock units were not exposed in this area. However, geologic logging of the shear pins installed for the slope repair to the north (Cotton & Shires, 1999) indicated that the geology in this area is composed of a non-homogeneous mixture of sandstone, sheared rock and greenstone of the Fransican formation.
- **Stations 28+00 to 32+75** - The repaired slope resulting from the 1997/1998 Polhemus Road landslide was noted along this segment of the alignment.

- **Stations 32+75 to 38+00** - The slope east of Polhemus Road along this segment of the alignment was noted to be steep and moderately vegetated with numerous displays of small/shallow slope failures and/or soil slips. Rock was exposed at various locations in this reach but was commonly mantled with colluvium or debris flows. The rock units exposed were primarily hard sandstone and chert. Review of the 1946 aerial photos did not indicate evidence for large-scale landsliding. However, this area may be prone to landsliding due to the steepness and height of the slope. A large hard rock outcrop was exposed at Station 39+00. This outcrop was steep and will likely be prone to rock falls.

- **Station 39+00 to End of Alignment** - From Station 39+00, the alignment diverges from the road and the slopes to the east. At station 42+50 the pipeline crosses Polhemus Creek. The stream bank slopes at the crossing were observed to be composed of the shallow greenstone. This assertion is based entirely upon the reconnaissance evaluation of the slope surface (i.e., bedrock fragments exposed in colluvial materials, shallow bedrock in road cuts, occasional bedrock surface exposures, etc.). On the west bank of Polhemus Creek, active lateral erosion of the creek had caused slumping and sliding of colluvial material overlying greenstone. A headscarp was observed about 20 feet to 15 feet above and west of the edge of the creek. Slopes in this area were typically about 45° steepening to near vertical near the creek.

Within the area evaluated, there are no geologic features or geologic hazards over the majority of the site that would prohibit construction of the pipeline in the center of Polhemus Road using open trench construction. However, steep topography and unstable ground was observed throughout much of the length of the alignment in the slopes east of, and above, Polhemus Road. Stability of slopes, with respect to their effect on the proposed pipeline, should be evaluated during final design of the project.

Appendix H

Photographs along the Final Alignment

The photographs included in this appendix were taken by Golder Associates, Inc. in February 2000 to support a conceptual evaluation of tunnel options for a new Crystal Springs Bypass Conduit. The photographs were presented in Golder's March 2, 2000 report entitled "First Progress Report on Alternative Design Study, 84-inch Crystal Springs Bypass Pipeline, Polhemus Road, San Mateo County, California."

APPENDIX H

PHOTOGRAPHS ALONG THE FINAL ALIGNMENT

Table F-1
Summary of Photographs

Photograph	Description/Location
1	Polhemus Lot from Entrance Gate on South
2	Polhemus Lot, View from Across the Road
3	South End of Alignment, Station 6+00 to 3+00 (approx.)
4	Station 14+00 to 3+00 (approx.)
5	Station 14+00 to 21+00
6	Rock Outcrop Near Station 12+00 (approx.)
7	Station 17+00, View South
8	Landslide Area South of Ascension Dr., Station 14+00 to 25+00 (approx.)
9	Boulder Nest in Landslide Area South of Ascension Dr.
10	Area between South End of Wall and Ascension Dr.
11	Turnout Area North of Wall
12	View from End of Wall
13	View South from North Portal Area
14	North Portal Area

Golder Associates Inc.

180 Grand Avenue, Suite 250
Oakland, CA USA 94612
Telephone (510) 239-9000
Fax (510) 239-9010

Golder Associates

983-7004

DATE: FEBRUARY 8, 2000
TO: PROJECT FILE – 983-7004
FROM: Kuantsai Lee

RE: DESIGN FLOW RATE, WATER INFILTRATION INTO TUNNEL TUNNEL OPTIONS, 84-INCH POLHEMUS ROAD BY-PASS PIPELINE

ASSUMPTIONS:

1. The excavated diameter of tunnel is 15 ft. by 15 ft.
2. Ground water table located at 3 ft. below road surface, about 40 ft. (max) above springline of tunnel.
3. Tunnel in Franciscan Formation of boulders, clay, gravel, and sand.
4. Estimate based on tunnels of similar dimensions in similar ground conditions.

MEASURED INFILTRATION RATES IN OTHER TUNNELS:

1. Toronto Subway: 16 ft. to 24 ft. internal diameter, water height at a maximum of 50 ft. above springline, in glacial till, sand, and clay, water flow 0.001 – 0.007 gal/ft^2/day.
2. Clyde Tunnel: 29.5 ft. internal diameter, water height at a maximum of 116 ft. above springline, through sandstone, sand, silt, and clay, water inflow 0.013 – 0.019 gal/ft^2/day.

ASSESSMENT:

Of the two cases cited above, the Toronto Subway Tunnel is more similar in dimension and water height while the Clyde Tunnel is more similar in ground conditions, but has over twice the water height then the present tunnel.

Expected water inflow is thus a little higher than the Toronto Subway, but less than the Clyde Tunnel. Propose to use 0.01 gal/ft^2/day.

With the proposed tunnel dimensions, the flow rate is 0.6 gal/day per ft. of tunnel. For the 720 ft. tunnel, the flow rate is 432 gal/day, or 0.3 gal/minute.

The capacity of a 12" diameter steel pipe at 4% grade is about 500 gal/minute, well above the expected rate.

Appendix J

Probabilistic Seismic Slope Deformation Analyses, 84-inch Pipeline

K Environmental Considerations
L Discussion of SFPUC FRP
J Probabilistic Seismic Slope Deformation

APPENDIX J

PROBABILISTIC SEISMIC SLOPE DEFORMATION ANALYSES
84-INCH POLHEMUS ROAD BY-PASS PIPELINE[1]

1.0 INTRODUCTION

Golder Associates Inc. (Golder) conducted this preliminary probabilistic seismic slope deformation analyses for the proposed new 84-inch steel Crystal Springs Bypass Pipeline (CSBP) to be located beneath the center of Polhemus Road to quantify the risk of damage to the proposed waterline that could result from an earthquake-induced ground movement. The purpose of this work was to attempt to provide a working tool quantify a relative risk of damage to a sufficient level for comparison to risk exposures for other pipeline alignments or for a tunnel alternative along the same alignment.

The analyses that follow were based on very limited geotechnical data as a detailed subsurface investigation of the site has not yet been completed. Of the 4,250 feet long alignment, subsurface geotechnical characterization has been conducted on about 400 feet of the alignment at the time this report was prepared. Most of this characterization was at the site of the repaired Polhemus Road Landslide. At the level that the analyses are presented in this study, they should also not be used as a basis for selection of an alternative alignment for the project, but serve to provide an understanding of the sensitivity of the input parameters to the results. In addition, this study addresses one risk that the new 84-inch CSBP will be exposed to. Other risks that should be characterized in detail for final pipeline design include:

- Potential pipeline damage from a landslide in the valley walls above the pipe not triggered by a seismic event, but triggered by future grading of the slopes or increases in the groundwater level under the slopes during or after heavy precipitation periods.
- Potential pipeline damage from a burst or break in the existing 96-inch concrete CSBP that could undermine the new 84-inch pipe.

2.0 TECHNICAL APPROACH AND ANALYSES PROCEDURES

A probabilistic seismic hazard analysis (PSHA) was conducted to determine the probability of exceedance of various peak ground accelerations (PGA) as a function of return period. Pseudostatic slope stability analyses were conducted over a range of soil strengths that might be expected to be encountered in the slopes east of the pipeline alignment to compute a probabilistic distribution of slope yield accelerations. A recorded acceleration-time history "base motion" was selected for the site and scaled to PGAs spanning the range of values determined from the PSHA. These scaled motions were then used in one-dimensional ground response analyses on one section of the proposed alignment thought to represent the worst case geometry with respect to slope stability. Horizontal equivalent acceleration-time histories were computed from the ground response analyses, and used with the range of computed yield accelerations in Newmark (1965) sliding block-type analyses to calculate a probability distribution of earthquake-induced displacements of the slope.

Prepared by Russ E. Browne, P.E., Golder Associates, May, 2000.

This approach is discussed in further detail in the following section.

2.1 Probabilistic Seismic Hazard Analysis (PSHA)

Seismic risk analyses were performed using Golder's in-house computer program GOLDRISK, and the seismogenic source parameter data presented in Table 1. GOLDRISK employs the seismic risk analytical procedure originally developed by Cornell (1968).

Table 1
Estimated Geologic/Geometric Characteristics of Key Potential Seismogenic Sources Located Within 100 km of the Project Site

Potential Seismogenic Source	Type [1]	Closest Distance (km)	Total Length (km)	Rupture Width (km)	Slip Rate (mm/yr) [3]	MCE (Mw) [2]
Hayward North	RL-SS	30	40	12	9	7.1
Hayward South	RL-SS	25	43	12	9	7
San Andreas	RL-SS	5	413	13	19	7.9
Green Valley	RL-SS	65	30	12	6	6.7
Concord	RL-SS	24	51	12	6	6.5
Rodgers Creek	RL-SS	31	64	10	9	7
Calaveras North	RL-SS	21	37	13	6	7.1
Calaveras South	RL-SS	64	48	5	8	7.1
San Gregorio	RL-SS	32	14	15	2	7.3

Notes:
1) SS – strike slip fault, RL = right lateral
2) MCE = maximum credible earthquake
3) The slip rates of the controlling Hayward Fault events (north and south segments) were reduced to 5 mm/year to account for fault creep.

The activity of seismic sources was modeled as a truncated form of the Gutenberg and Richter (1954) magnitude-frequency relationship given by:

$$Log N = a - bM$$

Where N is the cumulative number of earthquakes with magnitude greater or equal to M. The relation is truncated at the maximum credible earthquake (MCE) magnitude for each seismogenic source. The a- and b-values are coefficients that are dependent on the fault activity, and on regional seismicity, respectively. Based on the regional seismicity data, a b-value of 0.8 was assigned to all seismogenic sources. For each seismic source, the a-value was computed based on the minimum magnitude, maximum magnitude, fault length, the seismogenic width, and the seismic slip rate.

The earthquake ground motion attenuation relationships developed by Sadigh et al. (1997) for crustal rock sites were used to model the dependency of PGA on earthquake magnitude and source–to-site distance for the strike-slip faults considered in the analyses.

2.2 Pseudostatic Slope Stability Analyses

Golder then performed a pseudostatic, limit equilibrium slope stability analysis of section 7-7' using the Modified Bishop's procedure programmed in XSTABL (ISD, 1998). Section 7-7' is located about 60 feet north of the north end of the retaining wall constructed in 1998/1999 (Figure 1), and is thought to represent the worst case geometry that could be expected along the alignment for stability considerations. An idealized section depicting Section 7-7' is shown on Figure 2. The pseudostatic analysis considered the critical surface identified through performance of a static slope stability analysis. Because of the fine-grained fraction of the in-situ materials, and the rapid loading induced by an earthquake, the subsurface materials in the slopes east of the site were assigned undrained shear strengths. Based on the available laboratory strength test data, and our understanding of the subsurface materials at the site, undrained shear strengths were assumed to be normally distributed between 2500 and 3500 psf (mean strength of 3000 psf). This resulted in yield acceleration values in the range of 0.02 g to 0.26 g (mean yield acceleration of 0.15 g). Note that no actual subsurface information and laboratory strength data were available for Section 7-7' at the time this study was performed. The conditions used for this study were extrapolated from data developed for the Polhemus Road Landslide repair project south of Section 7-7'. This data and thus the cohesion value assigned are very approximate. Moreover, our analyses indicate that the yield acceleration is highly sensitive to this undrained strength of the slope materials. The estimated strength should be refined after a complete geotechnical investigation has been completed for the alignment and additional laboratory strength data has been developed.

2.3 One-Dimensional Ground Response Analyses

The program Proshake (EduPro, 1999), which is based on the program SHAKE (Schnabel et. al., 1972), was used to perform the ground response analyses. The ground motion recorded at the Diamond Bar seismograph station during the 1989 Loma Prieta earthquake was selected as the response analyses input base motion and scaled to 10 approximately equal-spaced PGAs ranging from 0.11 to 0.78g. Note that each of these scaled input motions corresponds to a different seismic risk level.

The analyses were conducted for a cross section located at the middle of the slope in Section 7-7' (about 160 feet from the toe and at a surface Elevation of 270 ft.). The shear wave velocities of the in-situ materials were estimated to vary from 600 feet per second (fps) at the ground surface, to 2000 fps at a depth of 120 feet. Materials below this depth were estimated to have a shear wave velocity of 2500 fps. Because of their generally fine-grained nature, the weathered materials at the site were assigned strain-dependent modulus reduction and damping relations developed by Vucetic and Dobry (1991).

A horizontal equivalent acceleration (HEA)-time history, which represents the average acceleration of the sliding mass, was computed for each of the 10 scaled input motions by dividing the shear stress-time history at the slide plane (65 feet from ground surface) by the vertical total stress at this location.

2.4 Newmark-Type Sliding Block Displacement Analyses

Newmark-type (1965) sliding block displacements were computed for each of the HEA-time histories using a seismic coefficient ranging from 0.02 to 0.25 g. The program YSLIP_PM, which is based on the integration of relative velocity procedure discussed by Franklin and Chang (1977), was used to calculate displacements. It was assumed that sliding occurs in the downslope direction only (i.e., upslope sliding was not permitted). The slope displacements computed by YSLIP_PM were plotted as a function of probability (from the PGA/probability relationship developed in the PSHA), and a best-fit power curve (R = 0.99) was fit to the data. Figure 3 presents this best fit curve, which shows seismically induced displacements as a function of exceedance probability over an assumed 50-year design life of the pipe. Superimposed on Figure 3 is the estimated damage potential to the 84-inch steel pipeline as a function of lateral displacement in a 100-foot length of pipe, as estimated by the Utilities Engineering Bureau of the SFPUC.

3.0 CONCLUSIONS

The results of analyses indicate that there is an approximately 1.5% probability that large (1-1/2 feet or greater) earthquake-induced slope displacement would occur at slopes represented by section 7-7’ during the next 50 years. The probability of experiencing 1 foot or greater of slope movement is 2.5 percent. These displacements could be assumed to occur over a 100 foot segment of the pipe. Figure 3 shows that displacements are generally sensitive to exceedance probabilities at low risk levels (about 10% chance of exceedance, and below). Sensitivity of deformations decreases with increasing risk level.

It is important to note that the displacements presented in Figure 3 are based on extrapolation of relatively little laboratory data. These analyses should be refined as additional laboratory strength data become available during final design.

4.0 REFERENCES

Cornell, C. A. (1968) “Engineering Seismic Risk Analysis: Seismological Society of America Bulletin, Vol. 58, No. 5.

EduPro Civil Systems (1999) “ProShake ground response analyses program user's Manual,” Redmond, Washington.

Franklin, A. G., and Chang, F. K. (1977). “Earthquake resistance of earth and rockfill dams,” *Misc. Paper S-71-17,* U. S. Army Waterway Experiment Station, Vicksburg.

Gutenberg, B., and Richter, C., 1954. Seismicity of the Earth and Associates Phenomena: Princeton University Press, Princeton, N.J.

Interactive Software Designs (ISD) [1994]. "XSTABL User's Manual." Moscow, Idaho.

Newmark, N. M. (1965) "Effects of earthquakes on dams and embankments." *Geotechnique*, 15(2).

Sadigh, K., Chang, C. Y., Egan, J. A., Makdisi, F. and Youngs, N. N. (1997) "Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data," Seismological Research Letters, Vol. 68, No. 1

Schnabel, P. B., Lysmer, J., and Seed, H. B. (1972) SHAKE – A computer program for earthquake response analyses of horizontally layered sites, *Report No. EERC 72-12,* Earthquake Engineering Research Center, University of California, Berkeley.

Vucetic, M. and Dobry, R. (1991) "Effect of soil plasticity on cyclic response," *Journal of Geotechnical Engineering*, ASCE.

Appendix K

Environmental Considerations

Appendix K

ENVIRONMENTAL CONSIDERATIONS[1]

The following discussion addresses potential environmental issues to be evaluated as part of this project and the necessary environmental clearances required before the project can be constructed.

4.1 California Environmental Quality Act Compliance

The California Environmental Quality Act (CEQA) is regarded as the foundation of environmental law and policy in California. It requires state, local, and other agencies subject to the jurisdiction of California to evaluate the environmental implications of their actions by requiring them to prepare multidisciplinary environmental impact analyses and to make decisions based on those studies' findings regarding the environmental effects of the proposed action. Furthermore, it aims to prevent environmental effects of the agency actions by requiring agencies to avoid or reduce, when feasible, the significant environmental impacts of their decisions.

CEQA is implemented in three phases. The first phase consists of preliminary review of a project to determine whether it is subject to CEQA. The second phase involves preparing an Initial Study to determine whether the project may have a significant environmental effect. Lastly, the third phase is to prepare an environmental impact report (EIR) if the project may have a significant environmental effect or a Negative Declaration or Mitigated Negative Declaration if no significant effects will occur.

A key decision in the CEQA process is determining the appropriate level of environmental analyses. The decision whether or not to prepare an EIR is generally discretionary with the Lead Agency (in this case, the City and County of San Francisco). To make this determination, it will be necessary to undertake pre-application consultation with the City's Office of Environmental Review (OER) staff. The purpose of pre-application consultation is for the Lead Agency to discuss the potential scope and content of the project's CEQA requirements. In March of 1999, the San Francisco Public Utilities Commission (PUC) submitted a Planning Department Environmental Review Application, project description, plan view and cross sectional drawing, and photographs of the proposed project (pipeline Alternative 1) to the OER and indicated that it believed a Mitigated Negative Declaration was required. The objective of the pre-application meeting will be to re-review the March 1999 submittal and discuss the four project alternatives and to identify as many issues as possible early in the permit process to encourage efficient and expeditious CEQA compliance.

Upon preliminary review of the four alternatives, it is anticipated that the focus of the environmental analysis will be on the following issues:

- Short-term (temporary) changes to or degradation of visual character;
- Nominal short-term construction air emissions associated with pipeline construction and excavation activities;

---

[1] Prepared by Tetratech, Inc., Environmental Consultant, April, 2000.

- Effects on biological resources, including special status species such as the federally endangered California red-legged frog and its habitat, the federally protected Evolutionarily Significant Unit (ESU) of the Central California Coast steelhead, and wetlands;
- Effects on subsurface cultural or historic resources;
- Short-term erosion, sedimentation, or siltation attributable to alteration of drainage patterns/course of San Mateo Creek and associated water quality degradation;
- Potential for landslides, subsidence, or liquefaction that could damage the pipeline or tunnel due to possible unstable soils;
- Short-term construction noise, including a temporary increase in ambient noise levels in the project vicinity above levels existing without the project;
- For the tunneling alternatives, potential for temporary generation of excess groundborne vibration or noise levels;
- Temporary increase in the number of construction-related vehicle trips; and
- Temporary traffic conflicts due to a single lane closure on Polhemus Road.

If, upon consultation with the OER (verify MEA), it is determined that an EIR would not be required for the project, then an Initial Study/Negative Declaration or Initial Study/Mitigated Negative Declaration will be prepared. The difference between a Negative Declaration and Mitigated Negative Declaration is that in the latter, appropriate measures are incorporated into the project description to mitigate potentially significant impacts identified. However, because this decision is at the discretion of OER staff, an alternative schedule that assumes an EIR is required also has been prepared (see Schedule Implications, below). Preparation of an EIR is more time consuming because it requires preparation of both a draft and final document and could require a more detailed analysis.

For the purposes of this study, it has been assumed that Alternative 1 will receive a Negative Declaration and that Alternatives 2, 3, and 4 may require preparation of an EIR.

4.2 Other Environmental Permits and Clearances

The four alternatives would require several permits due to proposed in-water construction activities in San Mateo Creek. The preparation and submittal of these permits is contingent upon early completion of a habitat assessment, which will include research and review of appropriate background materials and a field survey of the San Mateo Creek crossing. The findings of the habitat assessment will be incorporated into the CEQA document (see above), as well as the following permit applications:

4.2.1 California Department of Fish and Game, Section 1601 Streambed Alteration Agreement Application

Because the project involves diverting the natural flow of San Mateo Creek, notification must be given to the California Department of Fish and Game (CDFG) of such proposed activity. Notification requires preparation and submittal of a Streambed Alteration Agreement application, as well as consultation and possible negotiations with CDFG concerning appropriate mitigation measures. A draft CEQA document is required as an attachment to the Streambed Alteration Agreement application, but the Agreement will not be finalized by CDFG until proof of final CEQA approval (i.e., Final EIR certification or adoption of Negative Declaration) is provided.

4.2.2 Wetland Determination and US Army Corps of Engineers (Corps) Section 404/Section 10 Permit

Section 404 of the Federal Water Pollution Control Act authorizes the US Army Corps of Engineers (Corps) to issue permits for the discharge of dredged or fill material into waters of the United States at specified disposal sites. Nationwide permits (NWPs) are a type of general permit issued by the Corps and are designed to regulate with little, if any, delay or paperwork certain activities having minimal impacts. It is anticipated that NWP 12, Utility Line Backfill and Bedding, regulates discharge of material for backfill or bedding for utility lines, including outfall and intake structures, provided there is no change in preconstruction contours. A utility line is defined as any pipe or pipeline for the transportation of any gaseous, liquid, liquefiable, or slurry substance for any purpose. Material resulting from trench excavation may be temporarily sidecast (up to three months) into waters provided that the material is not placed in such a manner that it is dispersed by currents or other forces. The area of waters that is disturbed must be limited to the minimum necessary to construct the utility line.

State 401 water quality certification pursuant to Section 401 of the Clean Water Act, or waiver thereof, is required prior to the issuance of NWPs authorizing activities that may result in a discharge into waters of the United States.

4.2.3 Regional Water Quality Control Board (RWQCB) Section 401 Water Quality Certification or Waiver

Section 401 of the Clean Water Act requires that every applicant for a federal permit for an activity that may result in a discharge into a water body must request state certification that the proposed activity will not violate state and federal water quality standards. Water quality standards include beneficial uses of water (e.g., fish and wildlife habitat), water quality objectives (numerical and narrative limits on activities that impact water quality, including sediment, and any actions that generally increase in-stream toxicity and pollution), and the state's antidegradation policy, which requires that existing high quality waters be protected and maintained. The application must include a copy of the Corps Section 404 permit (see above) and the final CEQA document.

The Regional Water Quality Control Board (Regional Board) may waive the state's right to regulate the planned activity under its authorities if the project is determined to not violate water quality standards. Alternatively, the Regional Board may issue a certification based on a finding that the proposed discharge will comply with all pertinent water quality standards. To allow certification, conditions may be devised by the Regional Board to remove or mitigate potential impacts to water quality standards; such conditions must ultimately be included in the Corps Section 404 permit.

4.2.4 Biological Assessment and USFWS Section 7 Consultation

A Biological Assessment (BA) and informal consultation with the US Fish and Wildlife Service (USFWS) will be required pursuant to Section 7 of the Endangered Species Act

(ESA). Because San Mateo Creek supports both the federally endangered California red-legged frog and the federally threatened Central California Coast steelhead, consultation and coordination also will be required with the National Marine Fisheries Service (NMFS).

4.2.5 State Historic Preservation Office Consultation

Section 106 of the National Historic Preservation Act (16 USC 470-470w6), as amended (Public Law 89-515), and its implementing regulations (36 CFR Part 800.9 [a] and [b]), requires federal agencies to consider the effects of their actions on properties listed, or eligible for listing, in the National Register of Historic Places. Therefore, after completion of a cultural resources investigation (including record search and field survey) prepared as part of the CEQA documentation, appropriate documentation will be submitted to the State Historic Preservation Officer to satisfy Section 106 consultation requirements.

4.3 Environmental and Resource Agency Permitting Schedule Implications

Much of the information required in the application packages for the required permits is similar, and it is anticipated that a number of permit applications will be submitted to the applicable agency for review and processing concurrently. However, some permit applications can either not be submitted or in some cases approved without prior approval of another permit. For example, completion of the CEQA process (either adoption of a Negative Declaration or Notice of Determination of an EIR) is required prior to submitting the CDFG Streambed Alteration Agreement. In addition, RWQCB approval of the Section 401 water quality certification or waiver is required prior to Corps approval of the pre-discharge notification.

The sequence of permit submittals and approvals for this project along with their estimated duration is provided in the schedules herein enclosed. Option One assumes a Negative Declaration is required. Option Two assumes an EIR document is required. Both options assumed a June 2000 start, and would slip accordingly with later start dates.

Appendix L

Discussion of SFPUC Facilities Reliability Program (FRP) Phase II

January, 2000

Appendix M

Memorandum re. Valving